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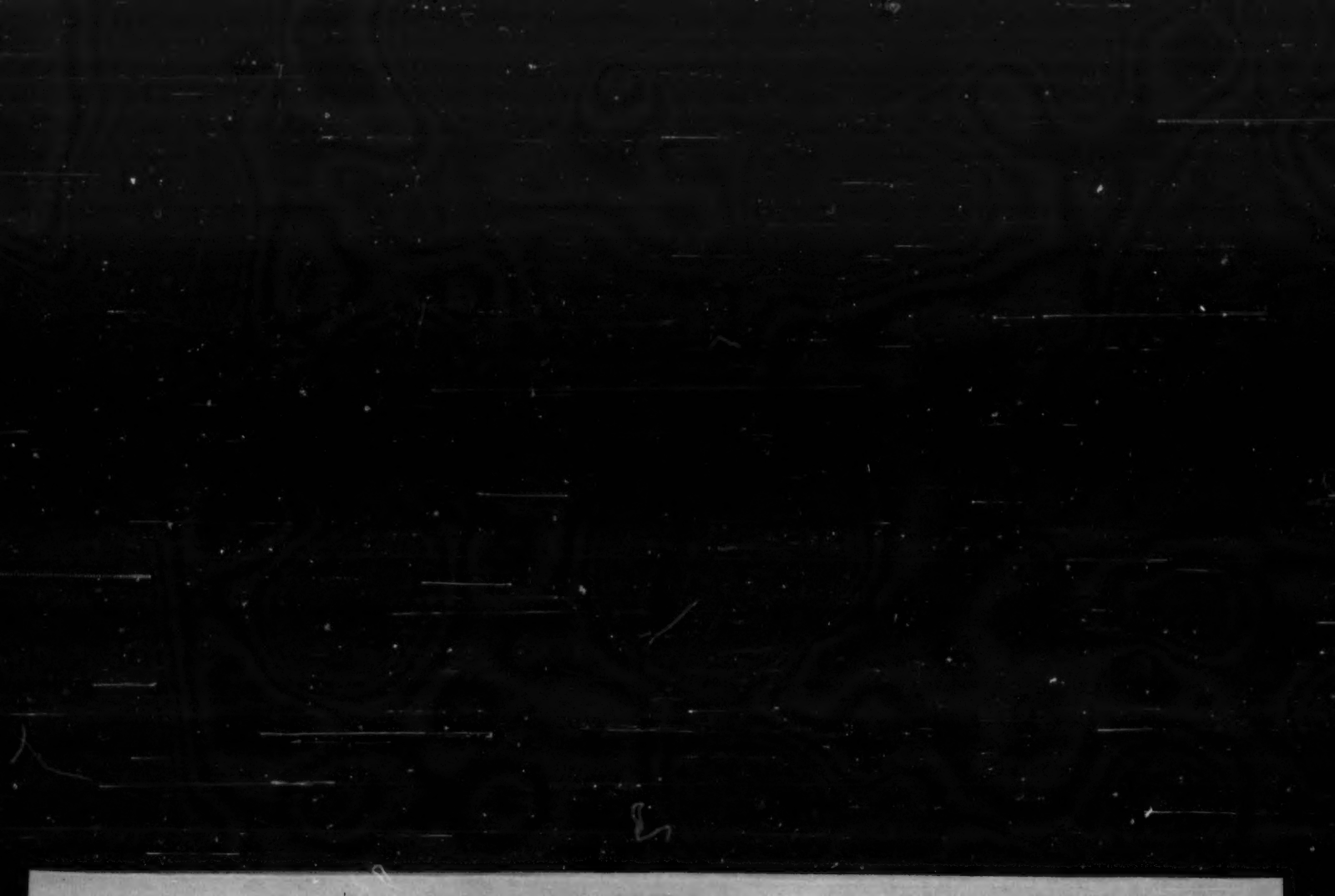

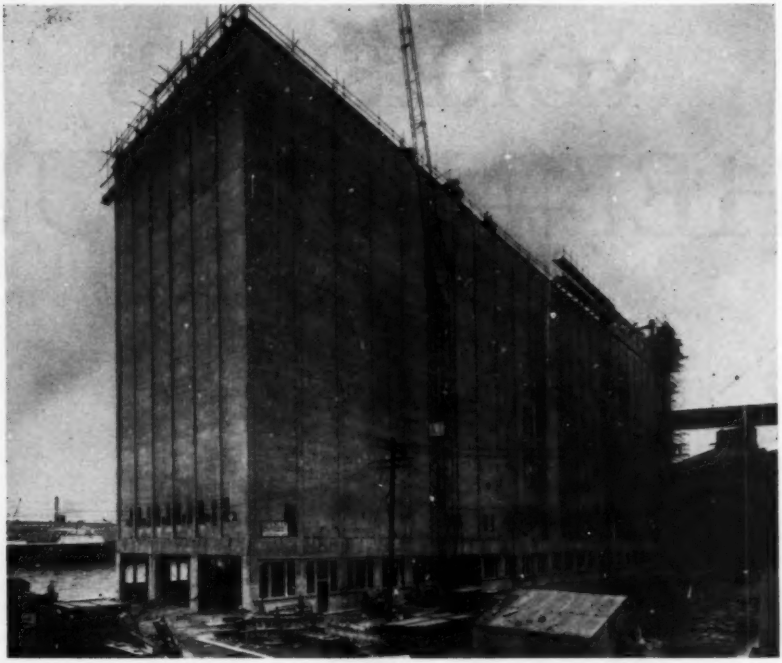

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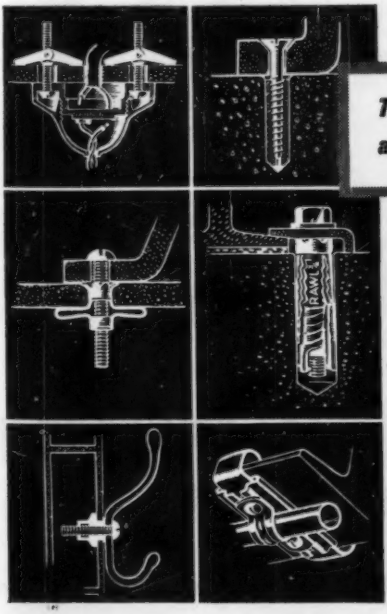

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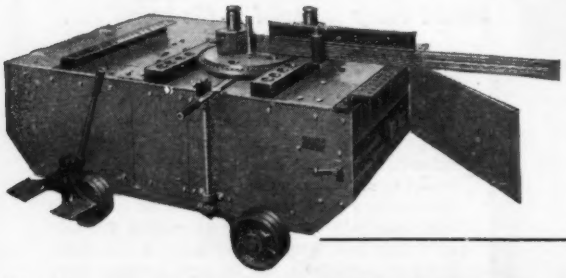

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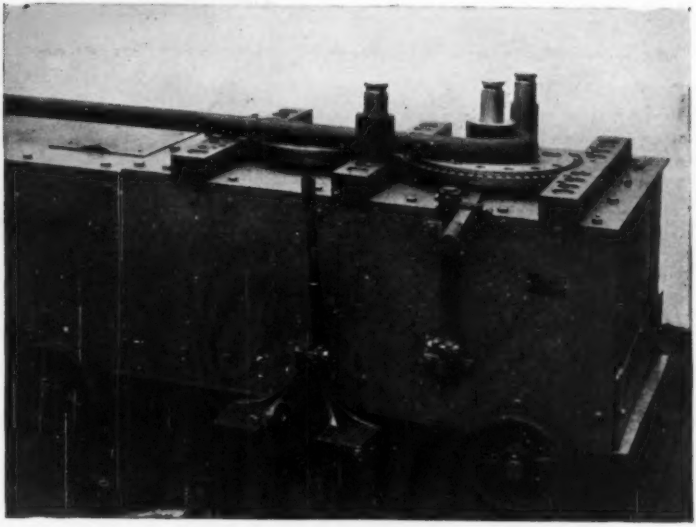

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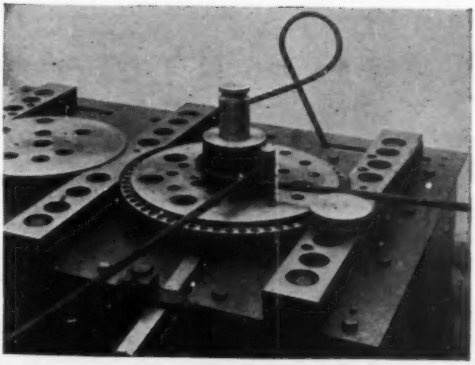

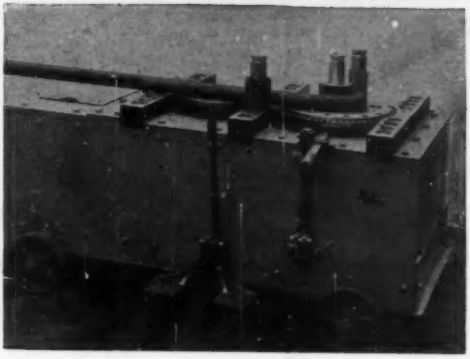

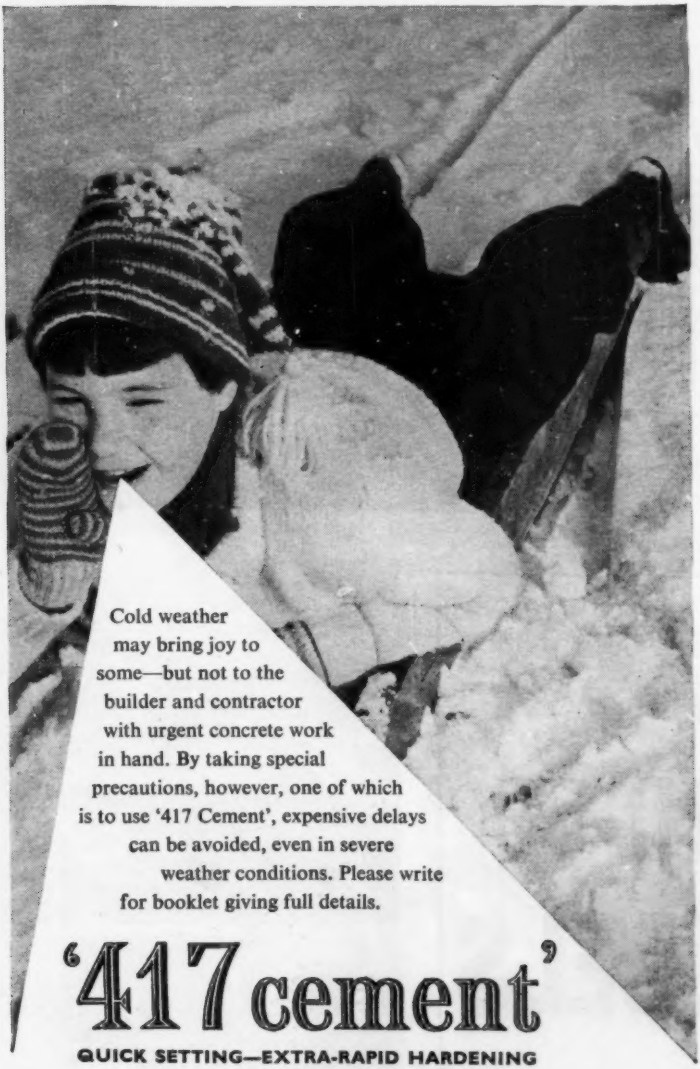

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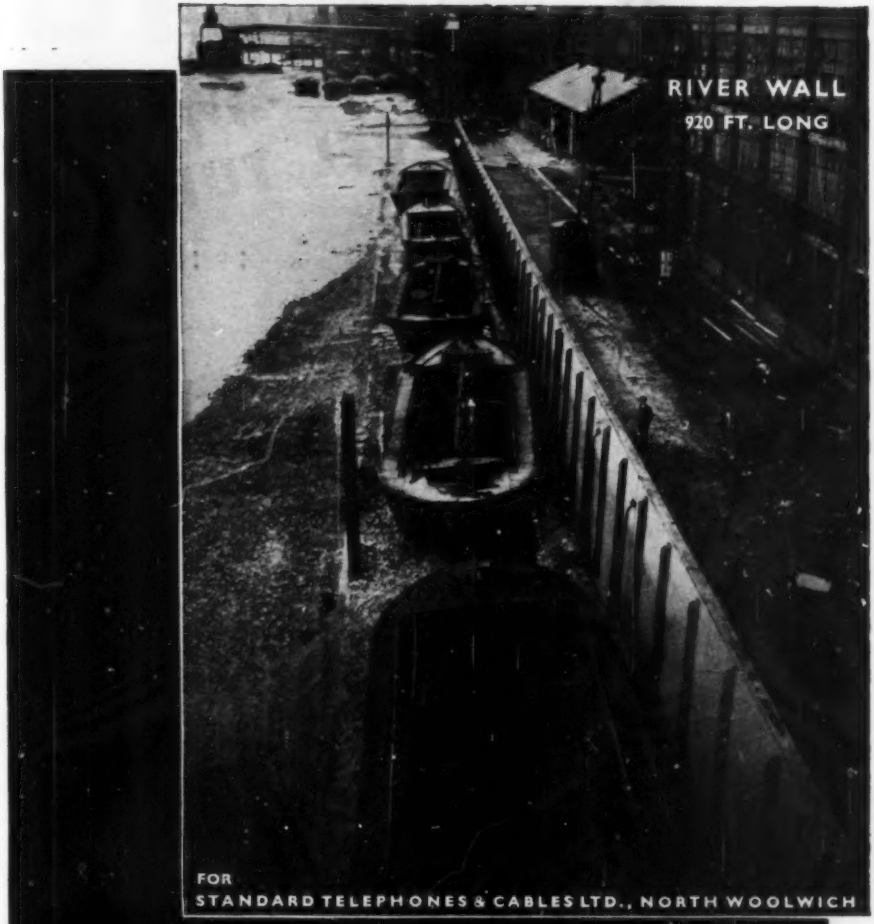

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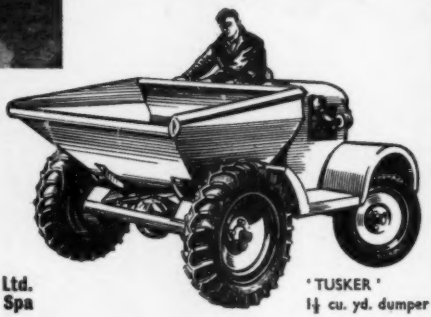

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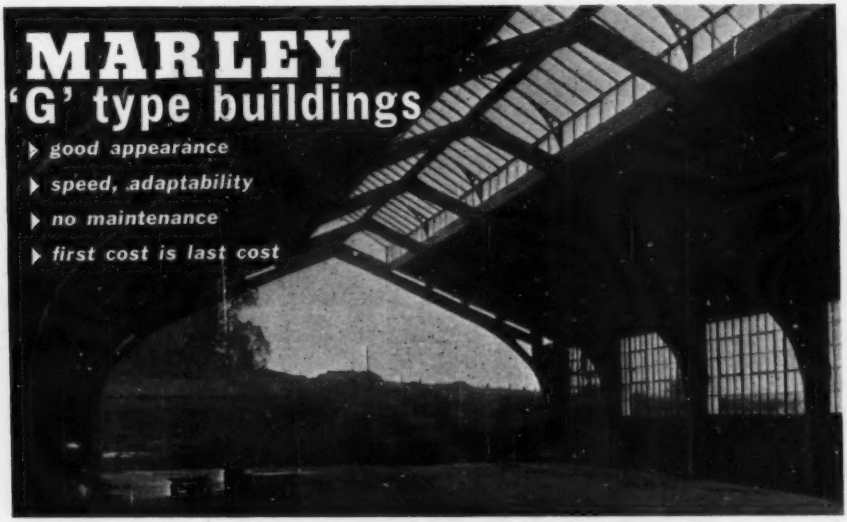

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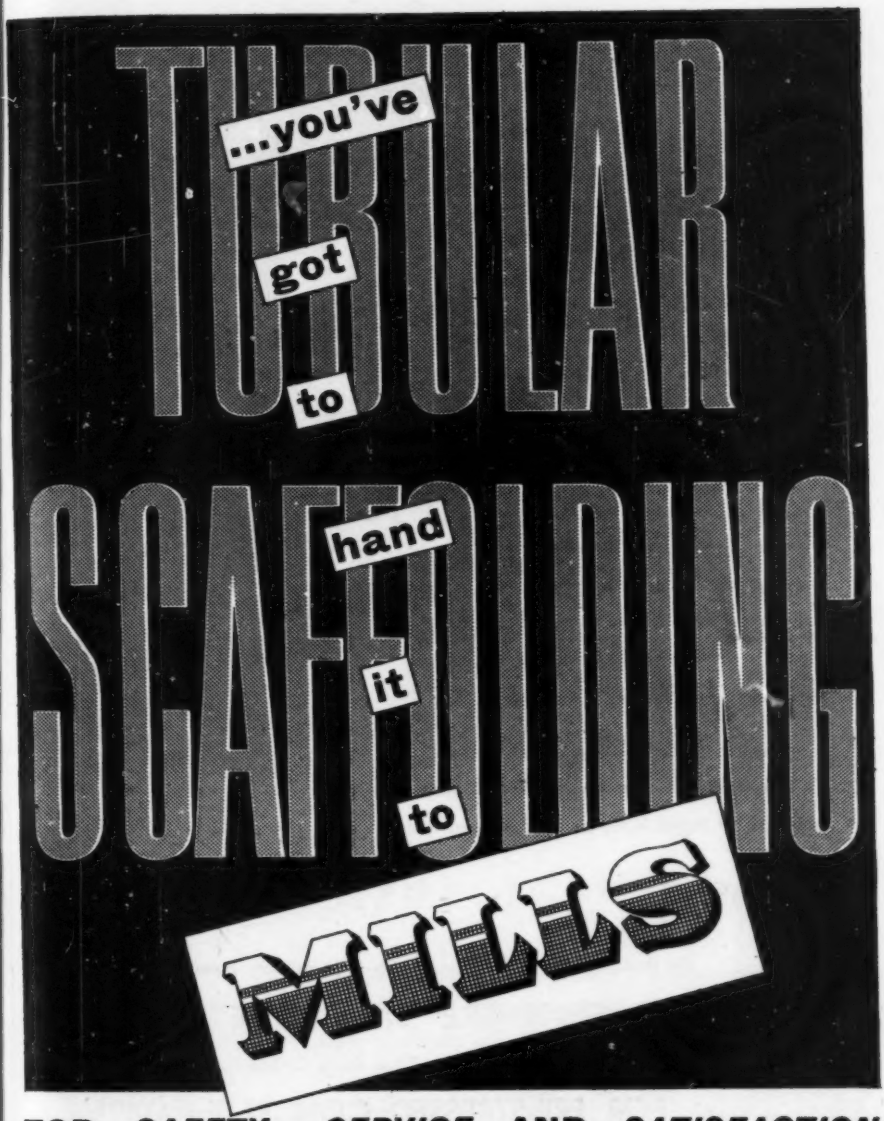

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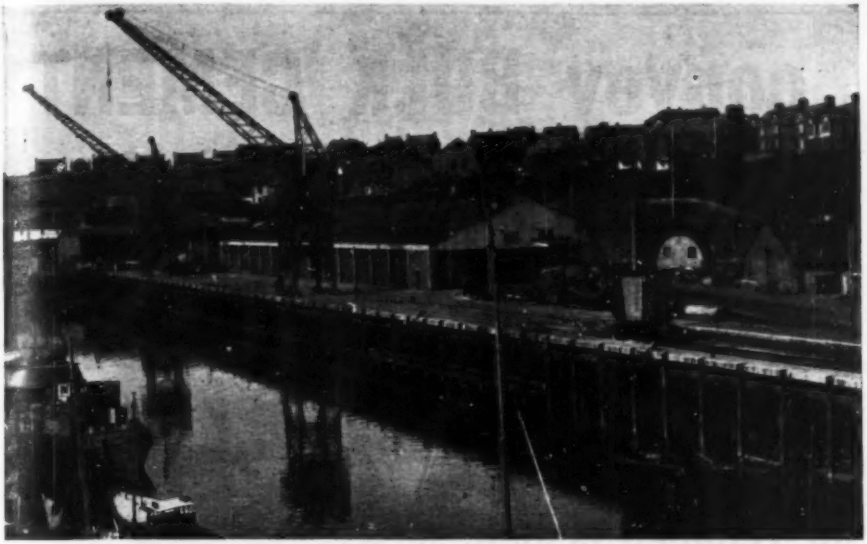

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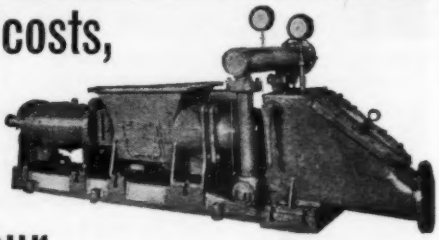

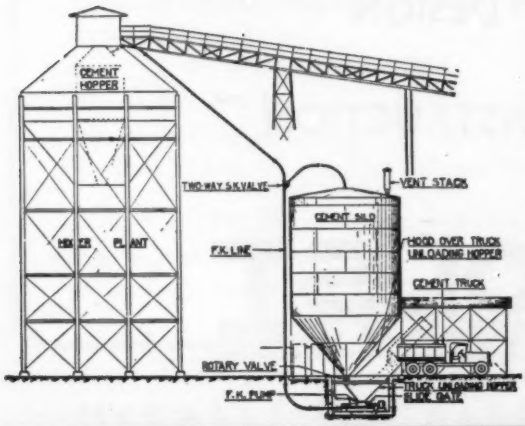

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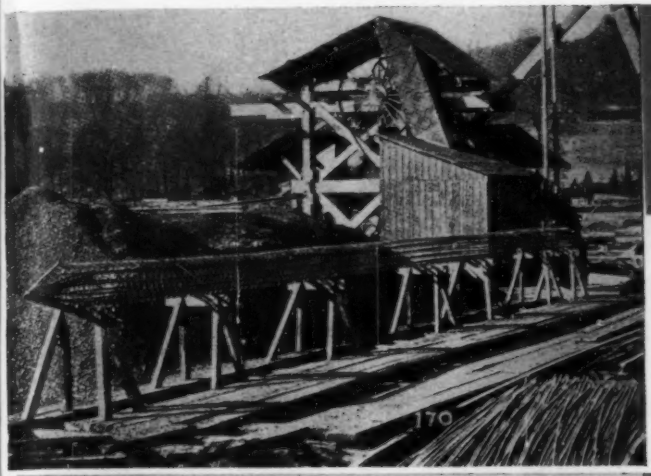

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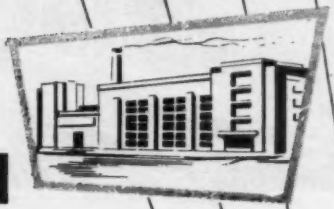

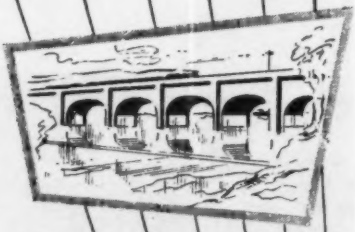

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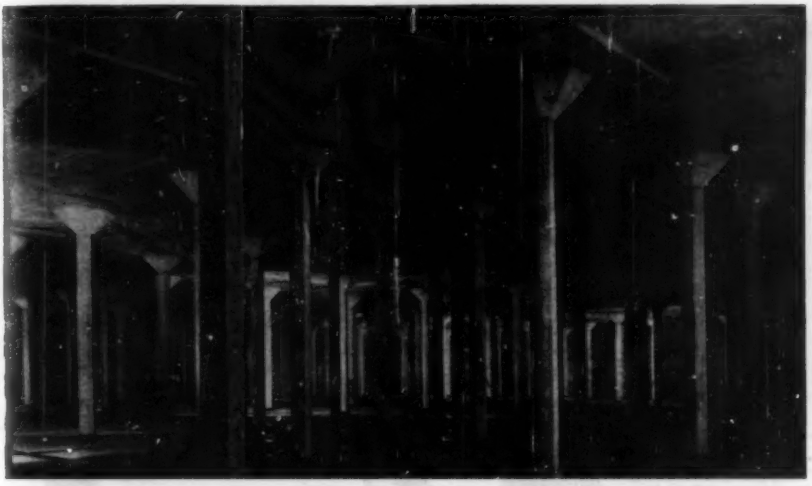

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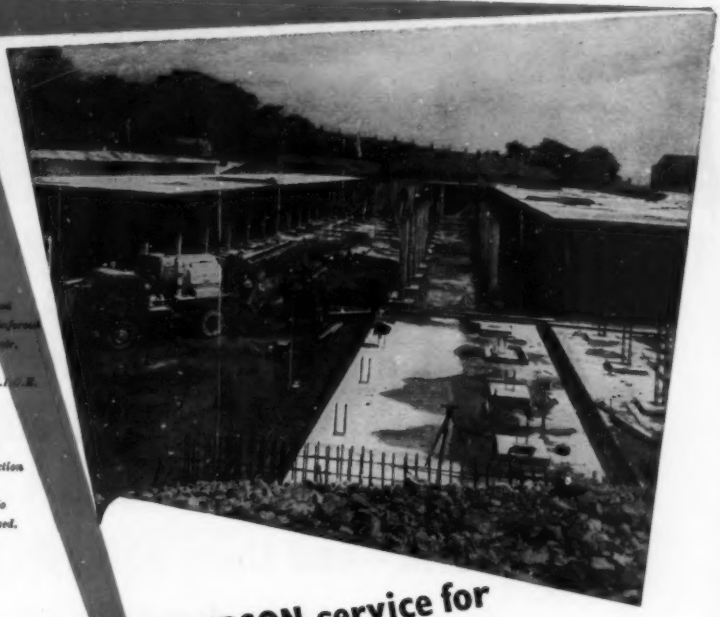

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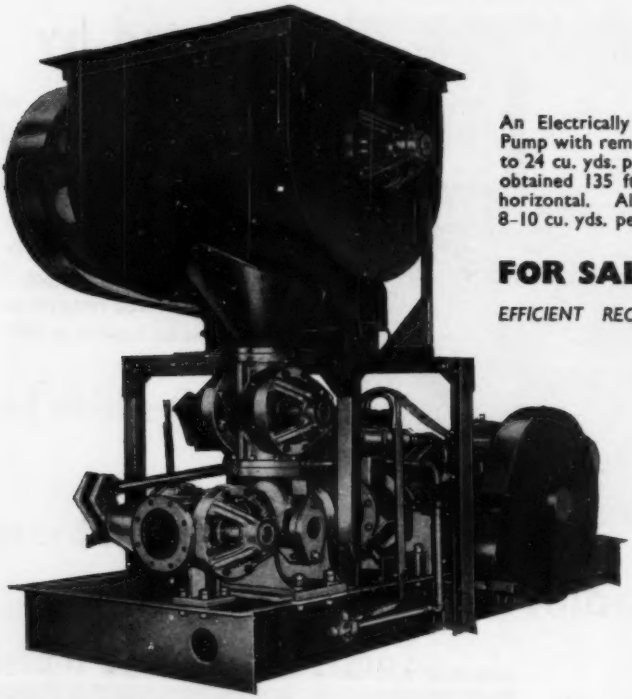

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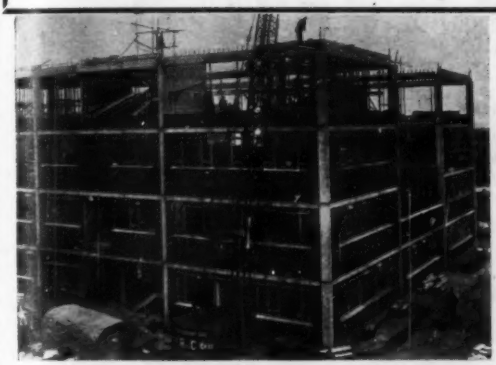

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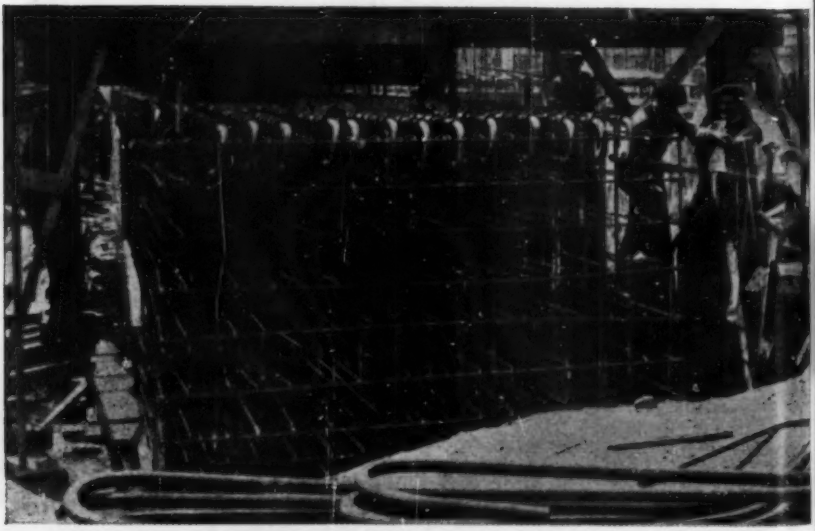

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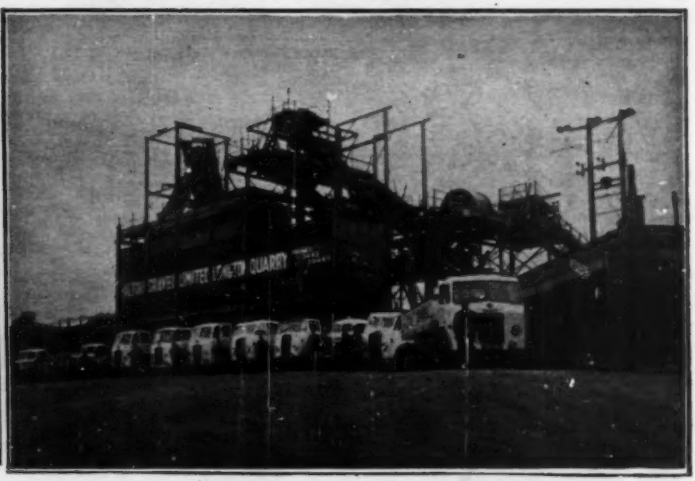

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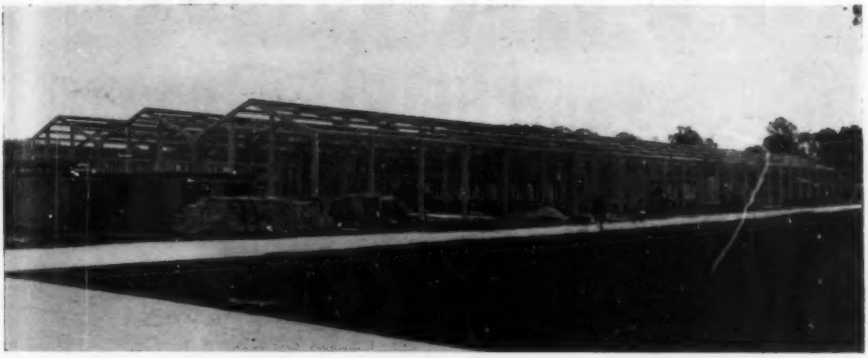

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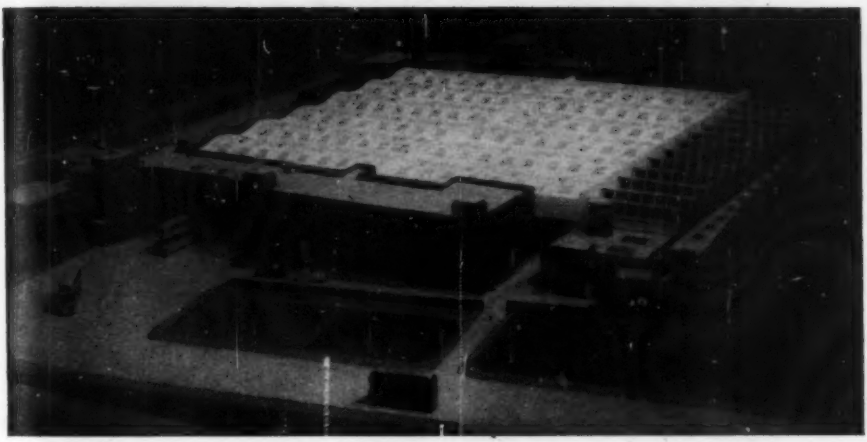

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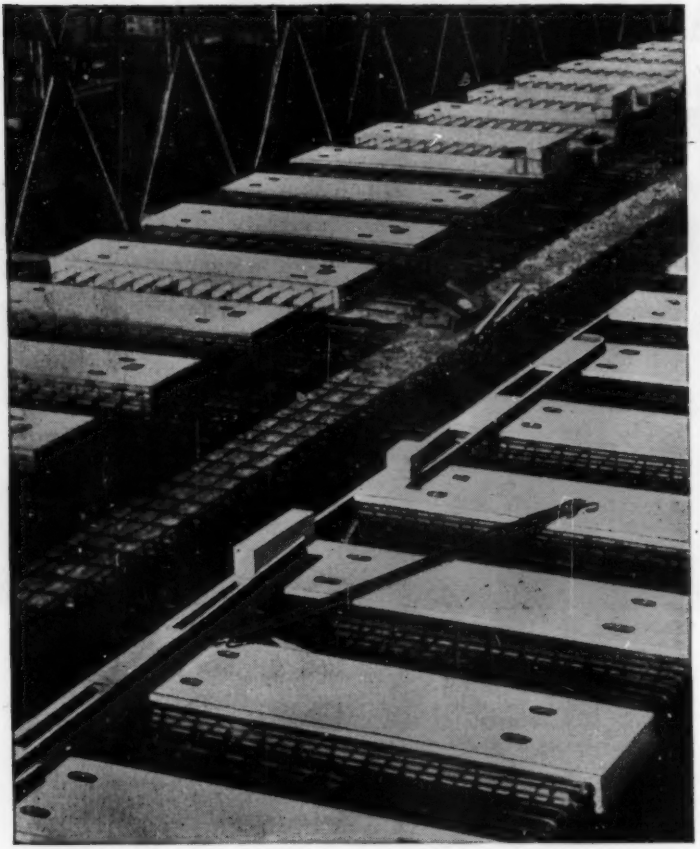

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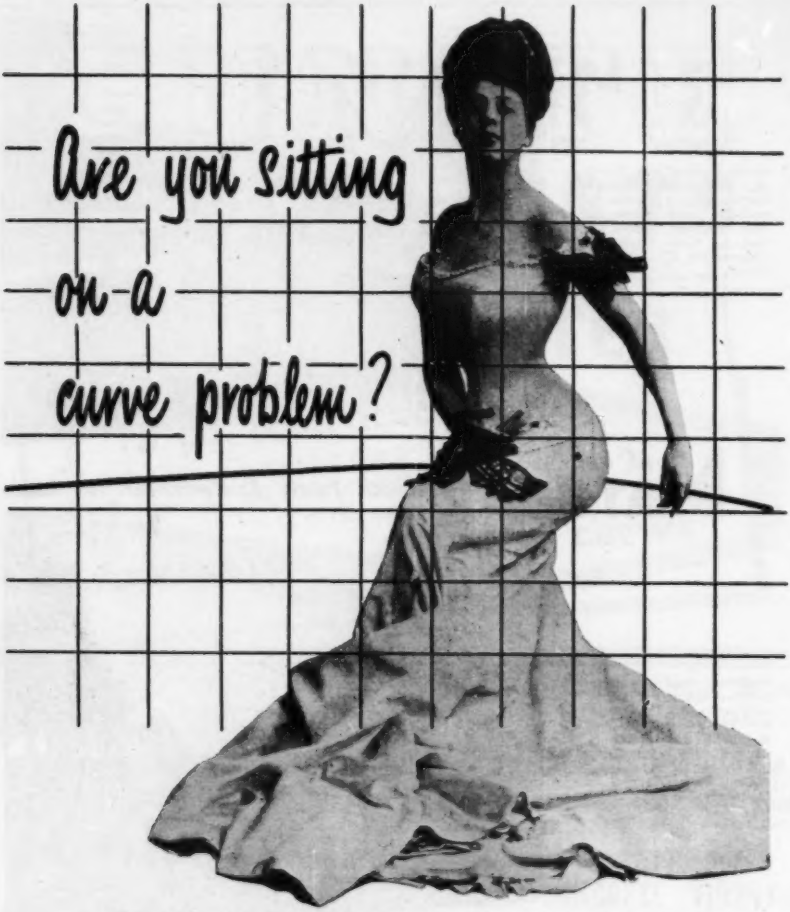

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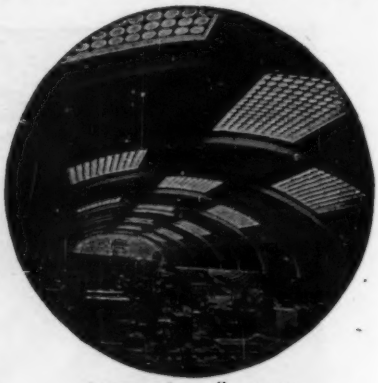

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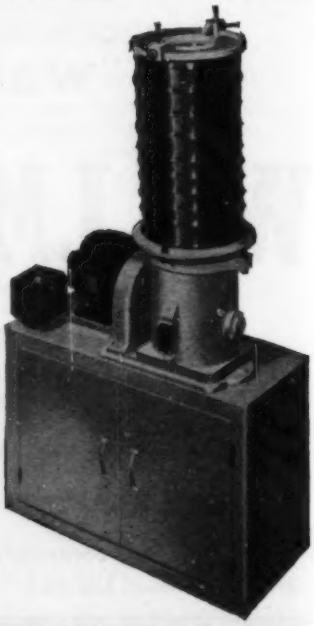

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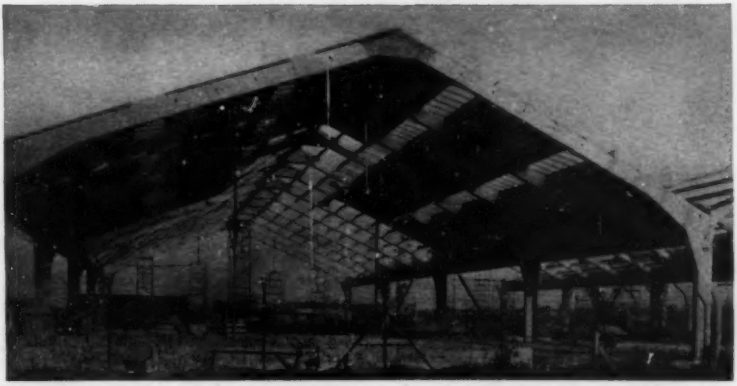

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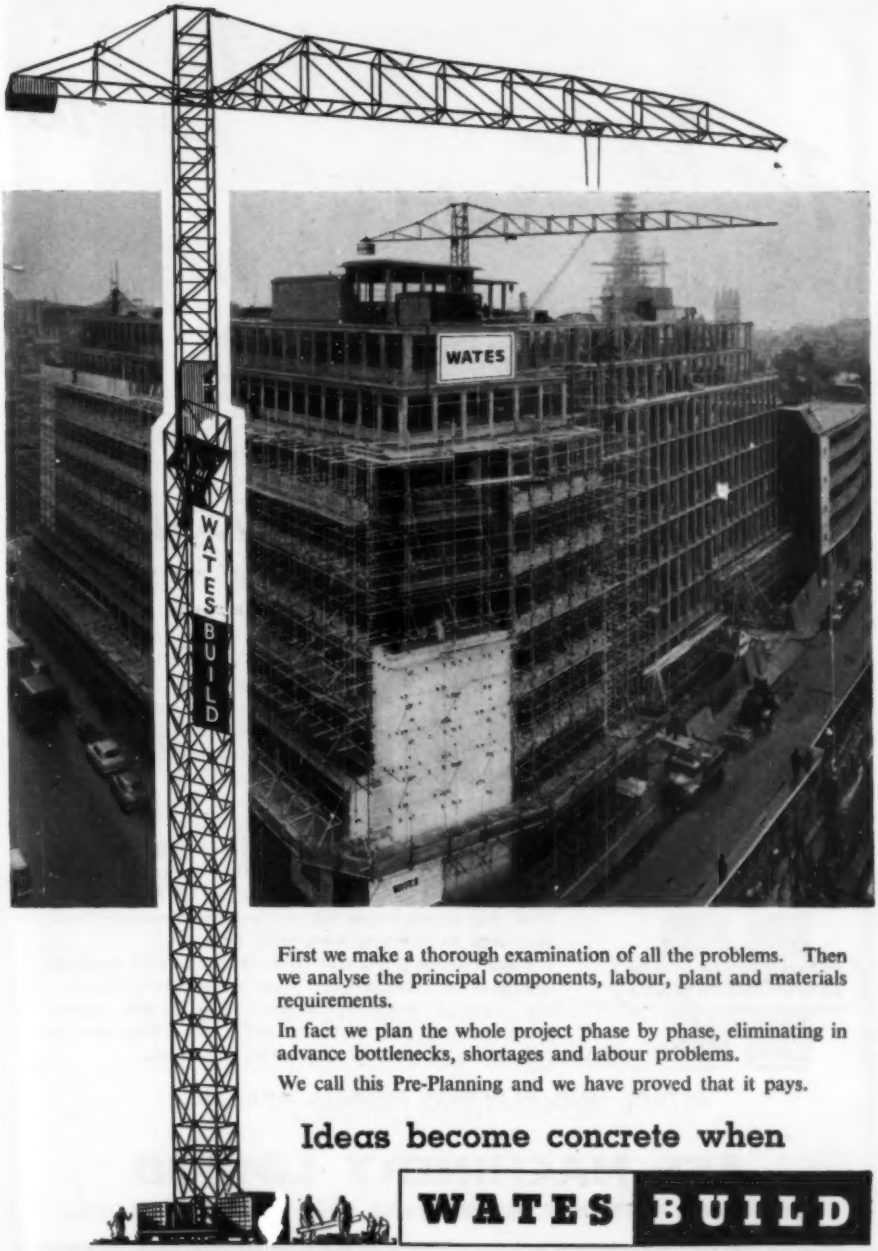

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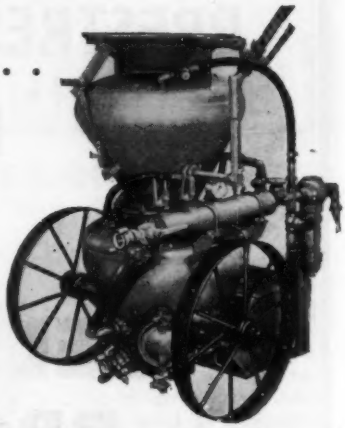

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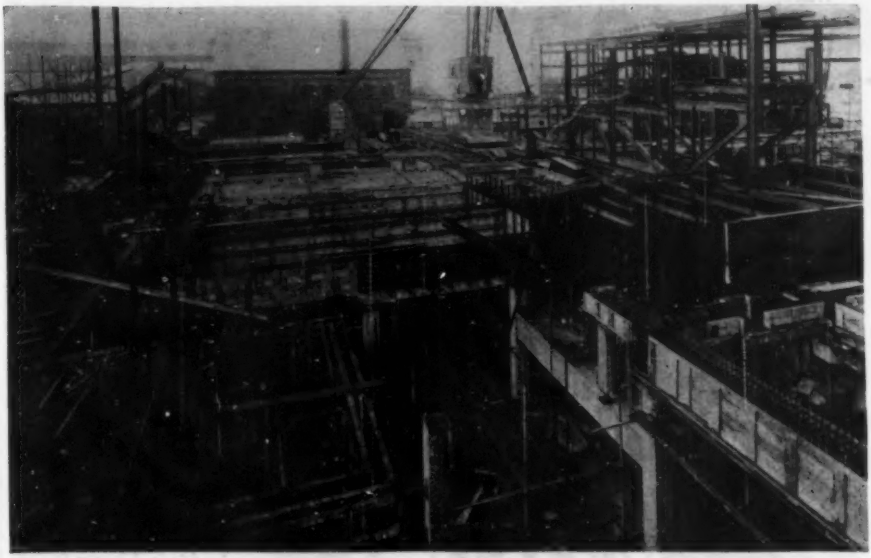

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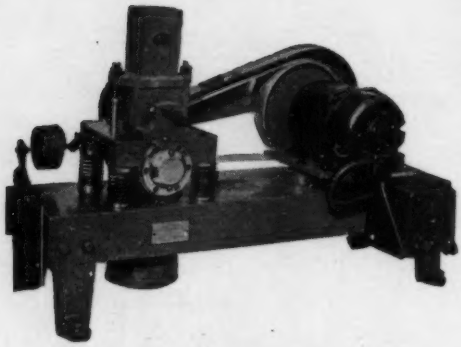

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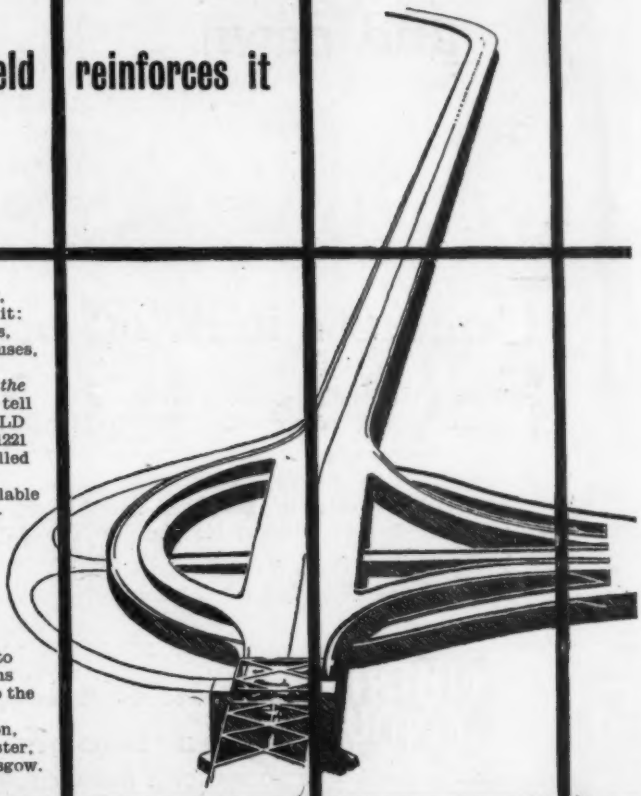

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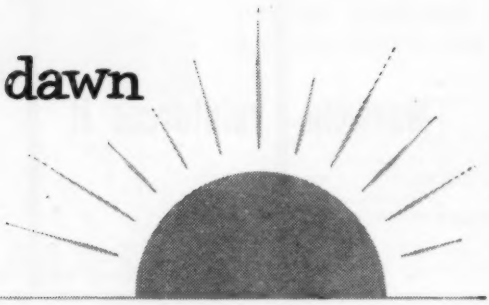

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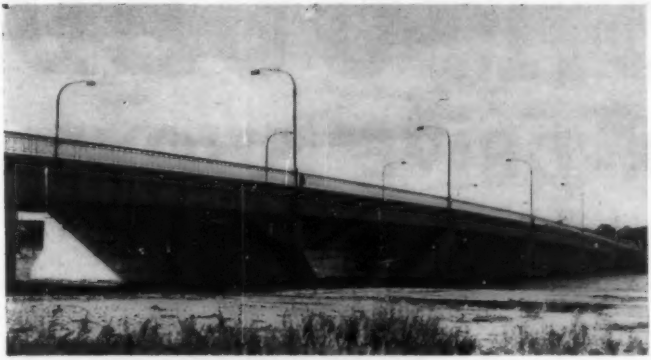
Precast Sections Prestressed together by Freyssinet System and Cable Ducts Colgrouted



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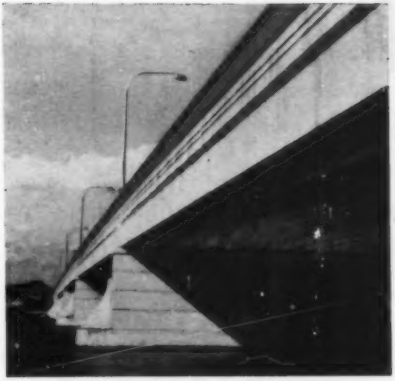

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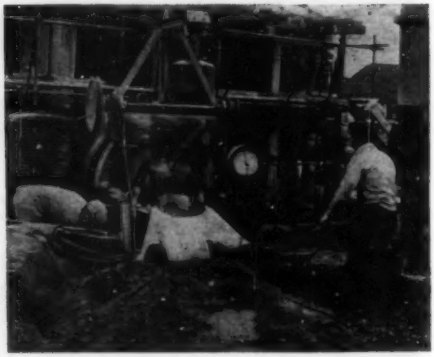

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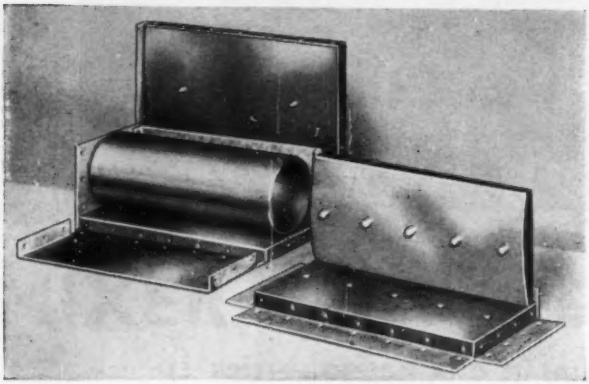
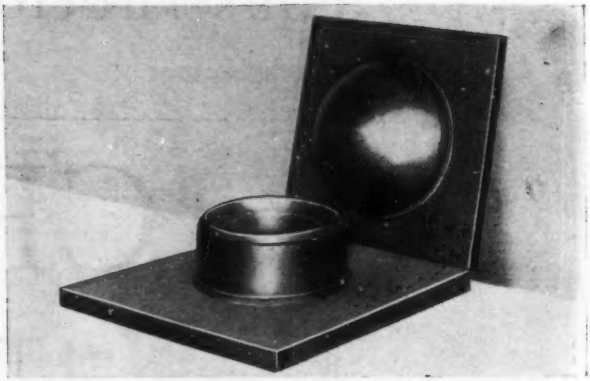
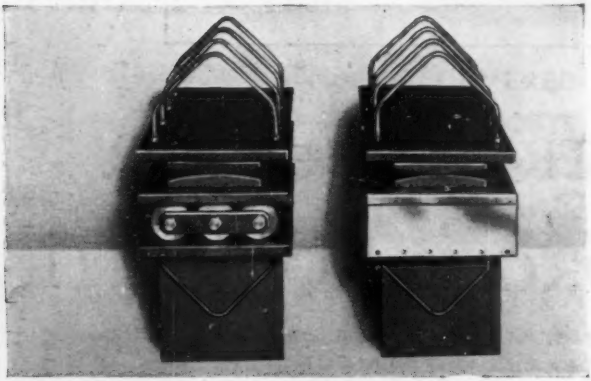


Illustrated are a few examples of various Bridge Bearings recently constructed by Westwoods.





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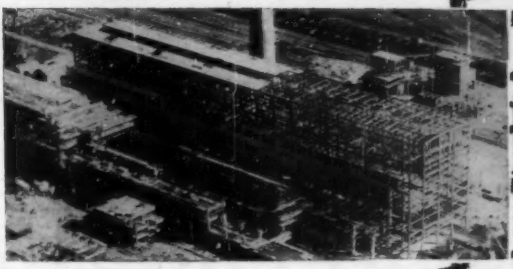

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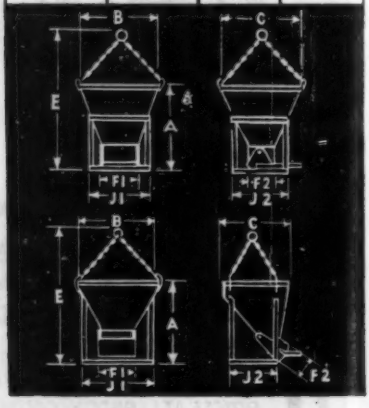

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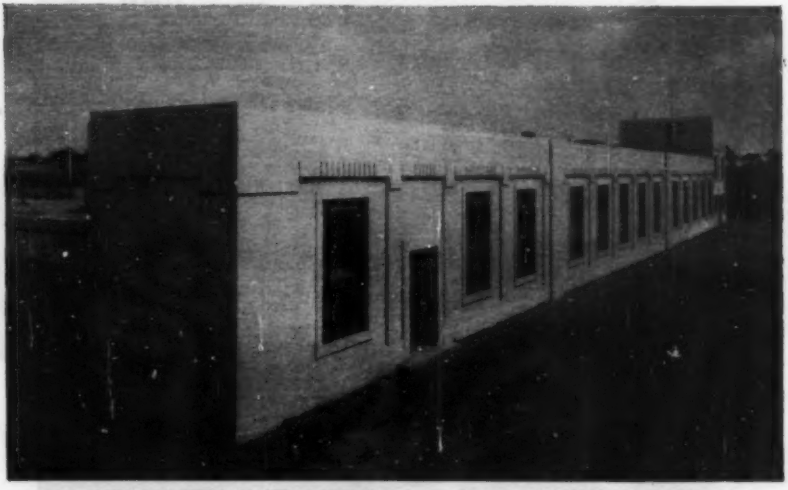

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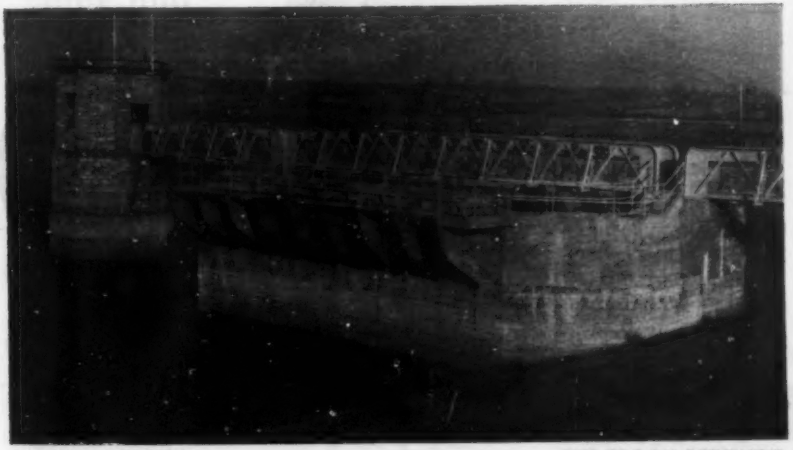

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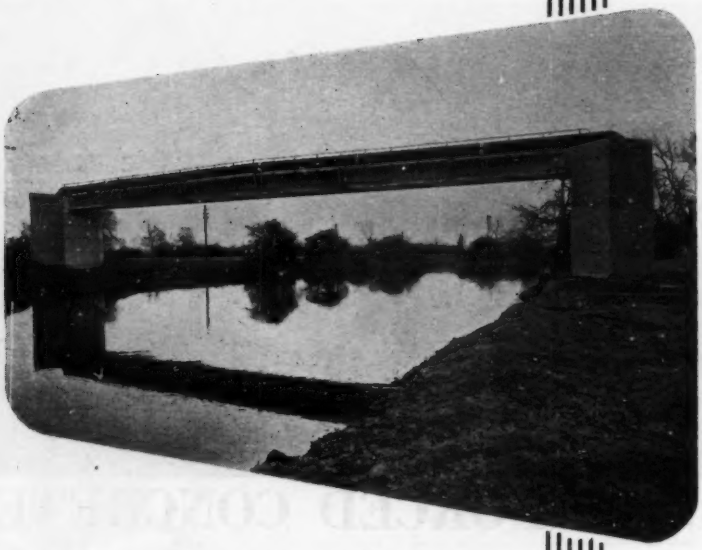

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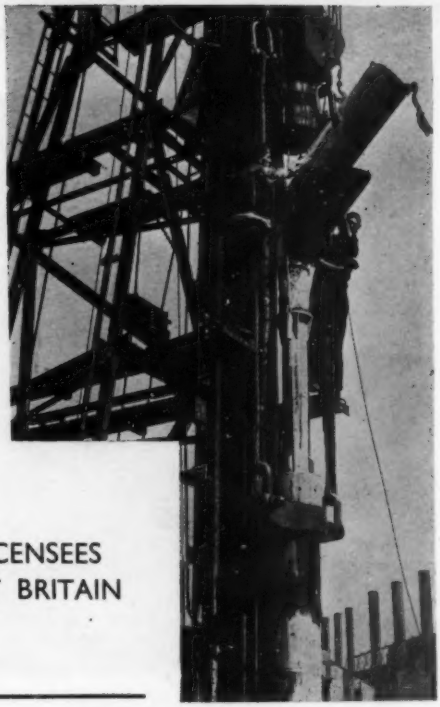

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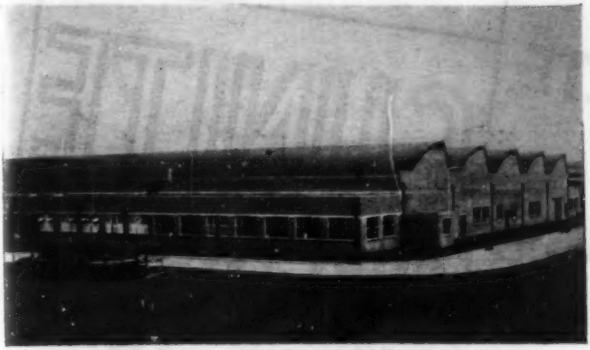

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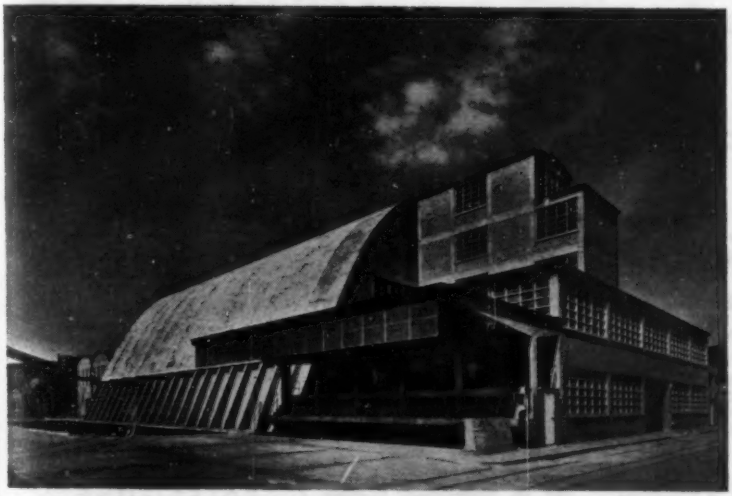

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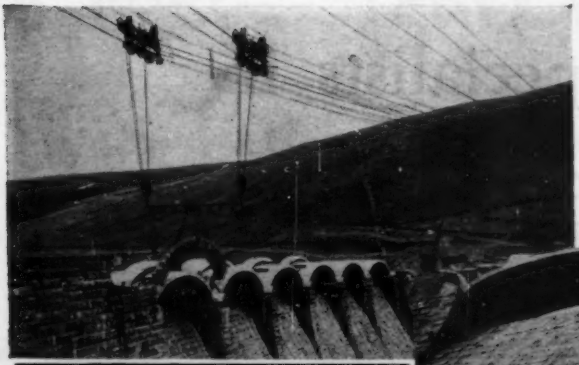

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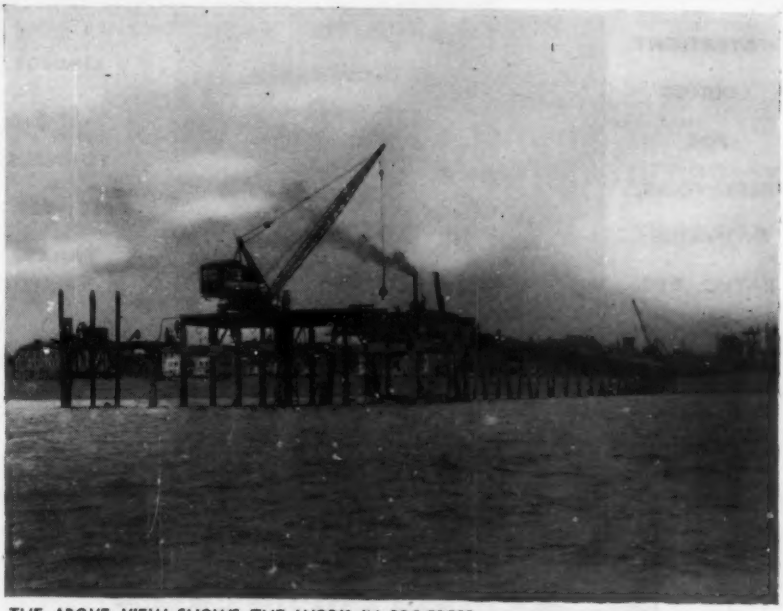

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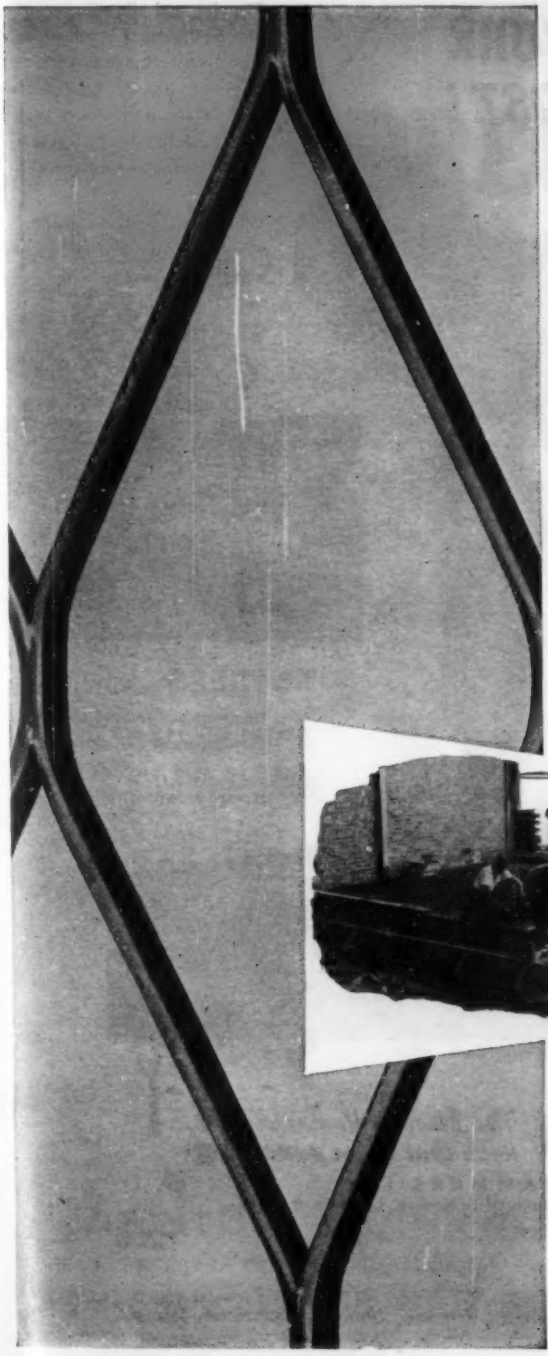

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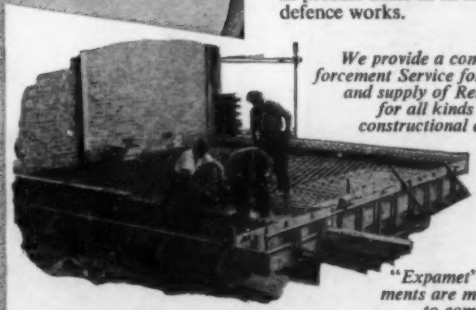

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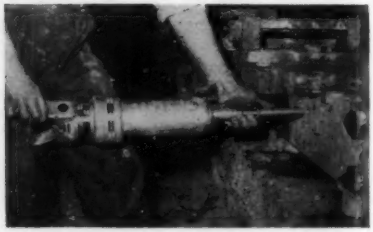

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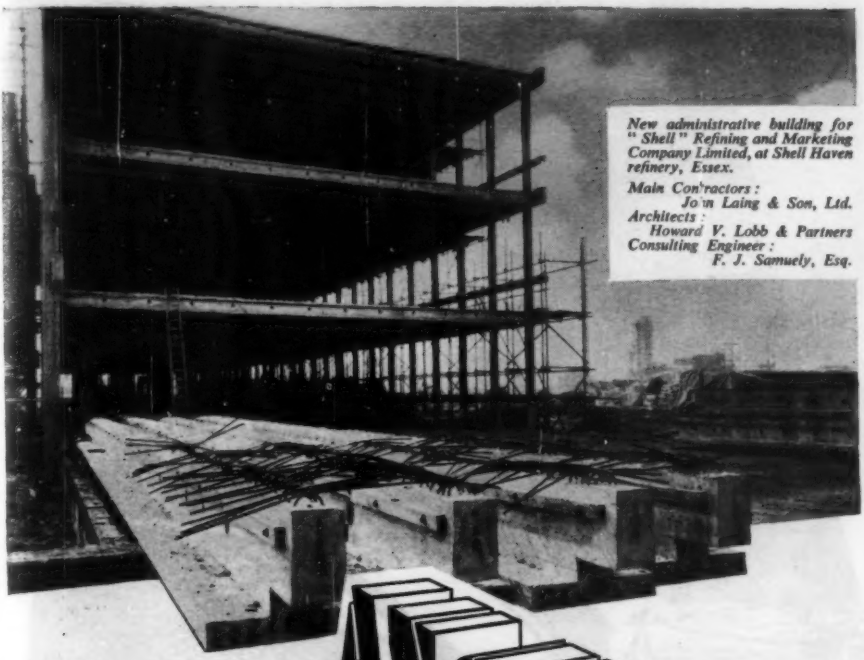

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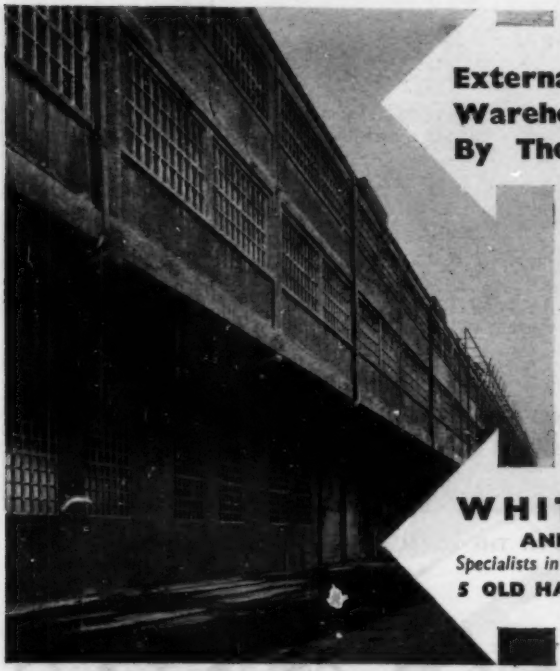

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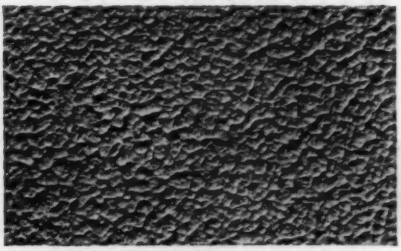

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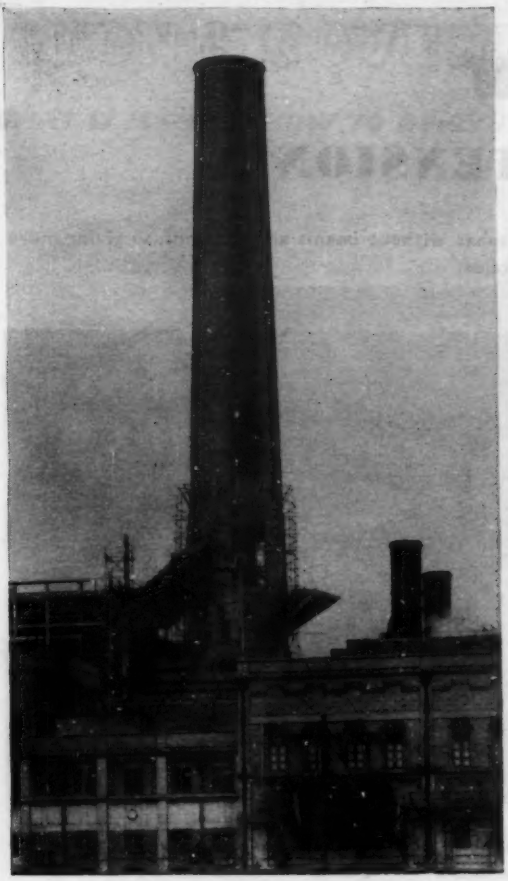

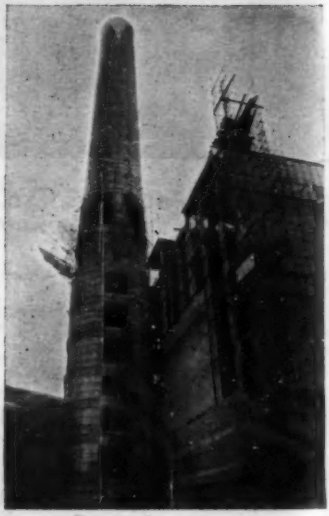

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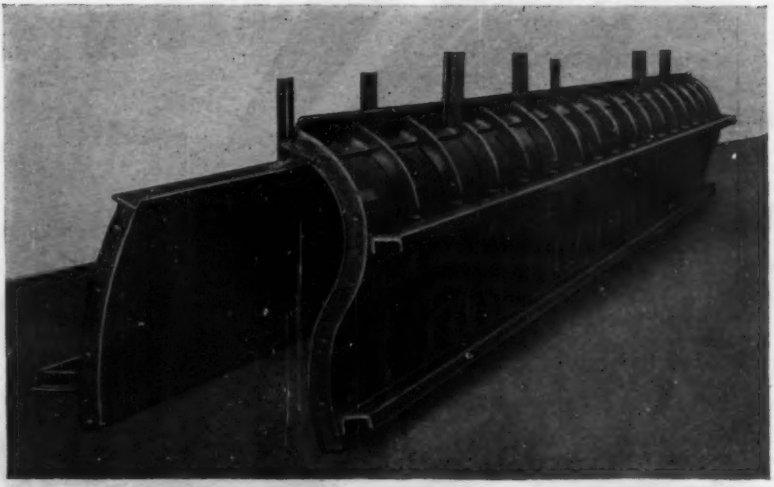

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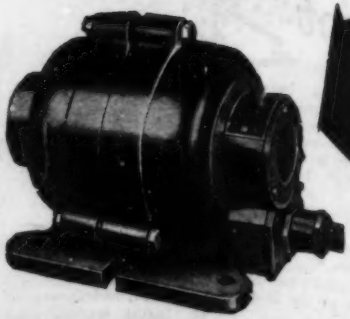

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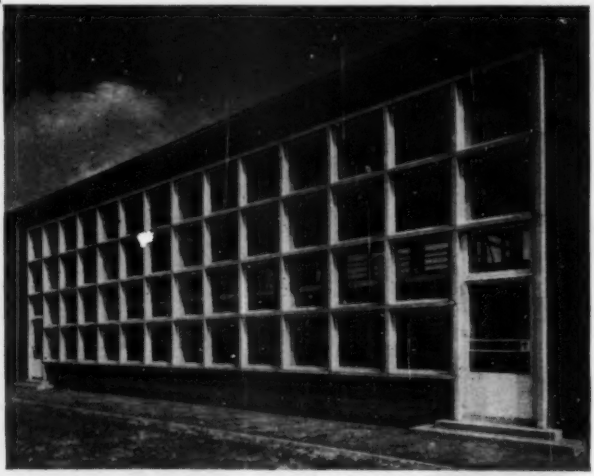

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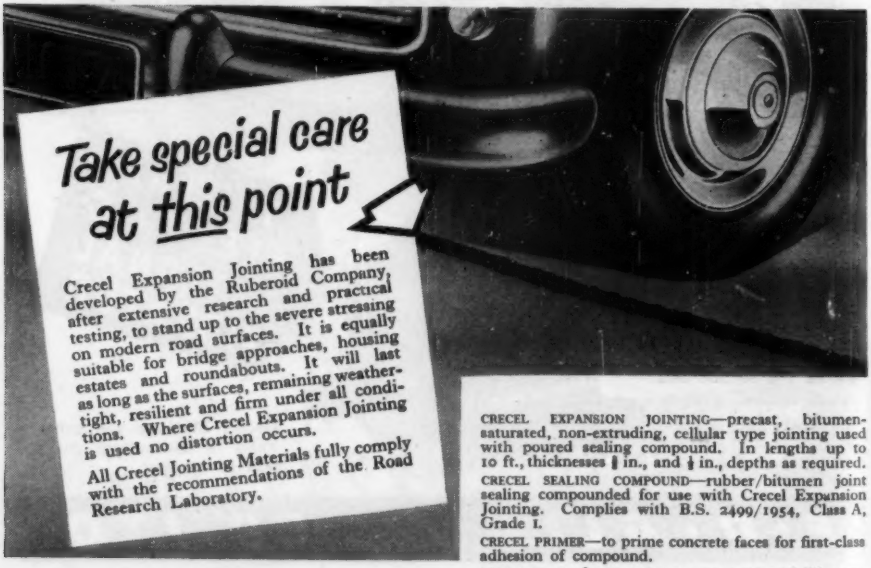
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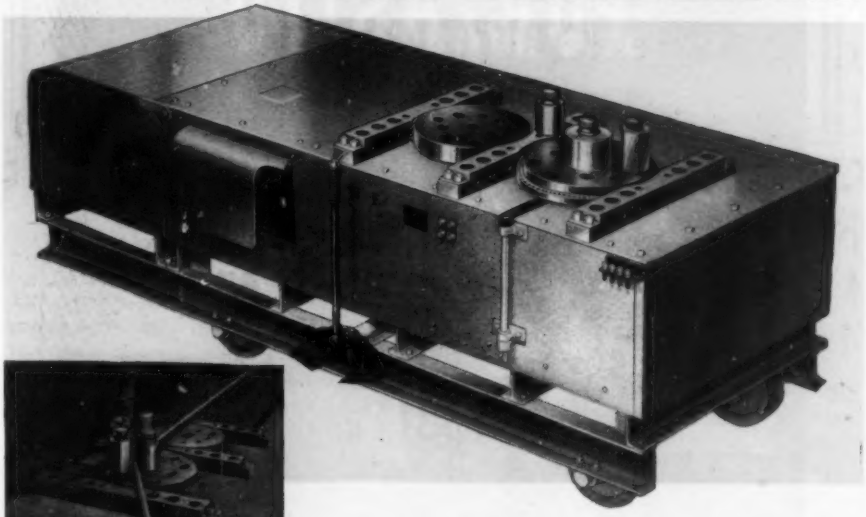

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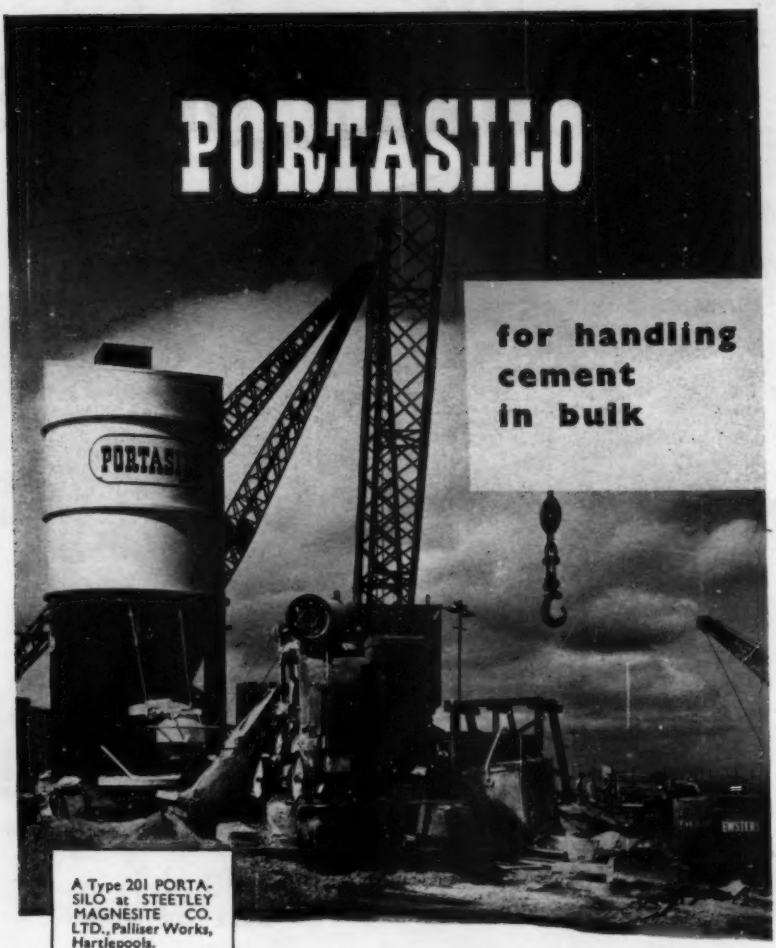

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Volume LII, No. 1. LONDON, JANUARY, 1957.

EDITORIAL NOTES

The Specification of Concrete.

THE need for an architect or engineer minutely to specify the materials and processes to be used in the manufacture of concrete, and to employ someone to see the work being done in order to ensure that his requirements are carried out, entails much office work and expense in addition to the cost of a representative on the site, and has naturally caused thought to be given to methods by which the onus for producing the required result is placed upon the contractor. Thirty or so years ago much was said and written about the advantages of specifying only the compressive strength required of concrete in different parts of a structure, and allowing the contractor to achieve these strengths as cheaply as possible. It was claimed that not only would this relieve the engineer of the responsibility and the labour of specifying the materials to be used, the proportions of the mixtures and the water content, the method of consolidation and so on, but that the cost would be reduced if the contractor were allowed to use the cheapest available materials, the cheapest mixture, and the cheapest methods of placing the concrete, provided only that the specified compressive strength was obtained. The suggestion found little favour amongst engineers who knew that all the properties of concrete were not necessarily related to its compressive strength. For example, in a thin section with much reinforcement it might be cheaper to use a fluid mixture rich in cement than a leaner mixture that needed much labour in tamping and spading; both concretes might have the same compressive strength, but the richer mixture would be much more liable to shrink and crack.

In more recent years the subject has been revived, but with precautions to ensure that the concrete has the required properties. An example is the specification for concrete now used by the Ministry of Works. The intention of this specification is that the contractor should, within limits and subject to the approval of the Ministry, use his own methods of producing concrete of the strength specified. So far, however, it seems to have met with little success, for towards the end of last year the Ministry announced that in the case of sixty-four out of seventy sites for which this specification was used the contractors had asked the Ministry itself to decide what were the cheapest concretes that would comply with the specification. There has therefore been little saving to the Ministry in office and site work in connection with the selection of materials

and the proportioning of concrete mixtures; indeed, the cost may have been greater. It is said that in the placing of 124,482 cubic yards of concrete on seventy sites, on which 117 different mixtures were used, up to May last year there was a saving of 3630 tons of cement.

The specification includes two main types of concrete, namely "Nominal Mix Concrete" and "Quality Controlled Concrete". In the case of "Nominal Mix Concrete" the required crushing strengths at twenty-eight days (in pounds per square inch) are 4000 for a mixture of 1 cwt. of cement, 2 cu. ft. of fine aggregate, and 4 cu. ft. of coarse aggregate; 3000 with 1 cwt., 2½ cu. ft., and 5 cu. ft.; 2500 with 1 cwt., 3⅛ cu. ft., and 6¼ cu. ft.; and 2000 with 1 cwt., 3¾ cu. ft., and 7½ cu. ft. The quantity of water must be the least necessary to produce these strengths and must be approved by the Ministry, and the Ministry must approve the aggregate the contractor proposes to use. The specified proportion of fine aggregate may be varied by up to one-third more or less than specified, and this seems to be the only matter left to the contractor's discretion.

The contractor is, however, recommended to use "Quality Controlled Concrete" if he can do so without extra cost. Here the contractor has an opportunity of making a choice but, as already mentioned, few have taken advantage of it. In the case of quality controlled concrete the proportions are specified by weight of cement to weight of combined aggregates. For a compressive strength of 6000 lb. per square inch at 28 days the minimum ratio of cement to aggregate is 1 : 6 for aggregate with a maximum size of 1½ in., 1 : 5 with ¾-in. aggregate, and 1 : 4½ with ⅜-in. aggregate; the maximum ratio of cement to aggregate is 1 : 3½ in all three cases. For a compressive strength of 4000 lb. the proportions are 1 : 9 with 1½-in. aggregate, 1 : 8 with ¾-in. aggregate, and 1 : 7 with ⅜-in. aggregate, with a maximum ratio of cement to aggregate of 1 : 5. For a compressive strength of 3000 lb. the proportions are 1 : 9 with 1½-in. aggregate, 1 : 8 with ¾-in. aggregate, and 1 : 7 with ⅜-in. aggregate, with a maximum ratio of cement to aggregate of 1 : 6½. In this "quality controlled" concrete the contractor can exercise his ingenuity in trying various proportions of fine to coarse aggregate, and of cement and water contents, within a very wide range, but there are lower and upper limits of cement contents that prevent the production of very rich mixtures that may be liable to excessive shrinking or very lean mixtures that may be lacking in durability. The mixture may be a continuous grading or it may be gap graded. The materials must be measured by weight and the concrete must be consolidated by vibration, and the degree of workability must be approved by the Ministry. Presumably the contractor is expected to make all the experimental mixtures and tests specified before submitting a tender, and preliminary tests on cubes and beams have to be made starting five weeks before concreting, using the plant to be employed in the work.

It is stated that the use of this specification has resulted in the saving of about 65 lb. of cement (say, 2*s.* 6*d.*) per cubic yard of concrete, but the extra cost of what is called "designing" the mixtures, of supervising the testing and the work is not stated. An interesting feature of the specification for quality controlled concrete is that in the preliminary tests twelve cubes must be crushed and at twenty-eight days must all have a strength of at least 15 per cent. above the strength specified. This is much more stringent than the requirements of No. 114, in which the average strength of only three cubes is acceptable.

Reinforced Concrete in Czechoslovakia.

By PROFESSOR KONRAD HRUBAN.
TECHNICAL UNIVERSITY, PRAGUE.

SINCE the beginning of the present century the development of reinforced concrete work in Czechoslovakia has been similar to that in neighbouring countries of Central Europe. Czechoslovakia lies on the watershed of the North, Baltic, and Black seas, and has generally a favourable geographical configuration, so that there is no need for spectacular dimensions such as bridges with very long spans or of great height. In the production of steel per inhabitant, Czechoslovakia now occupies the fifth place in the world; consequently, for large spans steel is preferred for all types of structures. The choice of material for structures must be related to the requirements of the national economy and, although the concrete structures may often be modest in size, the engineer strives to produce pleasing designs that will be a credit to him as well as to the material.

Bridges.

Some of the many bridges built in the past twenty years are typical of common practice in Czechoslovakia. A good example is the road bridge over the river Vltava shown in *Fig.* 1. This bridge is situated at a place where the water level will soon be raised to the springings of the main arch, which has the greatest span of any reinforced concrete structure in Central and Eastern Europe. The total length of the bridge is 1674 ft. The main arch has a span of 726 ft. and the other spans are 117 ft. The main arch has a rise of 141 ft. and is 6 ft. 7 in. thick at the crown. It was completed in 1945. The arrangement of three arches in the central span considerably reduces the weight of the structure; so far as the writer knows it is unique of its kind and is an interesting solution of the problem of supporting a road on tall arches. The system is, of course, highly statically indeterminate.

A large railway bridge is shown in *Fig.* 2. While concreting the main arch a gap of 3 ft. 8 in. was left at the crown, and this was widened by means of jacks

Fig. 1.—Road Bridge over the River Vltava.

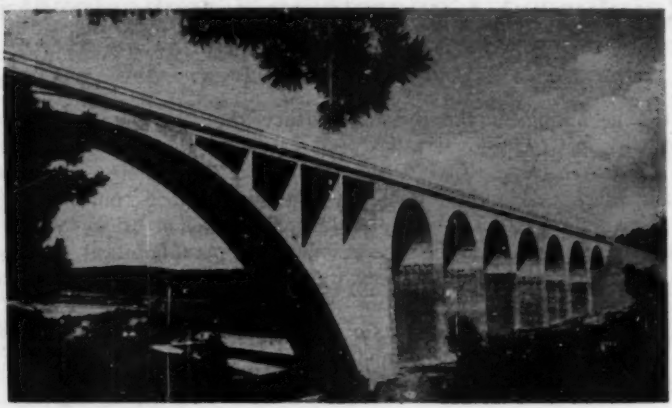


Fig. 2.—Railway Bridge near Brno.

Fig. 3.—Bridge over the River Lužnice.

Fig. 4.—Bridge over the River Váh.

after the concrete had hardened in order to raise the crown and release the centering; the space was then filled with quick-hardening concrete. A reinforced concrete railway viaduct 3000 ft. long has just been completed near Prague with fifteen spans of 175 ft. each.

If deck girders of sufficient depth are provided, a considerable slenderness of the arch of a bridge can be achieved (*Fig.* 3). This bridge over the river Lužnice, with a total length of 550 ft., was built in 1934 and 1935. The main arch has a span of 270 ft. and thicknesses of 1 ft. 10 in. at the crown (that is 1/146 of

Fig. 5.—Hotel in Prague with Tower 246 ft. High.

Fig. 6.—Commercial Building at Gottwaldov. Height 243 ft.

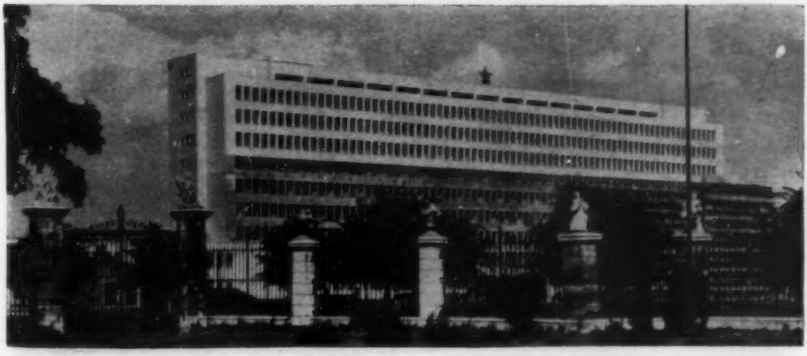


Fig. 7.—Office Building at Bratislava.

the span) and 3 ft. 4 in. at the springings. In 1948 a bridge of the same type to carry a motor-road was completed; the deck is supported by, and acts with, four arches with spans of 296 ft.

A structural system similar to that of the famous bridge over the Tiber * in Rome (built by Hennebique in 1911) has been used for the bridge shown in *Fig.* 4, which has just been completed. The structure appears to be an arch with a clear span of 369 ft. and a rise of 28 ft., compared with a span of 330 ft. and a rise of 33 ft. in the case of the bridge at Rome. The rise-span ratio is less than 1/13 and true arch action is practically eliminated. The structure has a cellular cross section consisting of a curved slab 10 in. thick at the centre of the span and increasing to 24 in. at the springings connected by vertical longitudinal walls to the deck which is 10 in. to 14 in. thick. The structure acts as a box-girder fixed in the abutments.

Structures.

Examples of tall buildings with reinforced concrete frames are shown in *Figs.* 5 to 8. The frame of the office building at Bratislava (*Fig.* 7) comprises both steel and reinforced concrete members. The gymnasium on the top of the residential flats shown in *Fig.* 8 has cylindrical shell roofs.

The chimney at Gottwaldov (*Fig.* 9) is the tallest reinforced concrete structure (460 ft. high) in Czechoslovakia. The base is 70 ft. high, and contains filters for collecting the fuel-ash which is stored in silos below. *Fig.* 10 shows the

* See *Fig.* 2, page 37, of this journal for January, 1956.

Fig. 8.—Flats at Gottwaldov with Circular Columns.

construction of the shaft with continuously-moving steel shutters from which the concreting platform was suspended. The rate of construction was 6 ft. a day.

Examples of the use of reinforced concrete in the construction of buildings are given in *Figs.* 11 to 14. The post-war industrial reconstruction has nearly doubled the rate of building, and new structural systems, economising in both labour and material, had to be introduced. This resulted in the precasting of structural members on a large scale. Also, different types of shell roofs have been investigated, and detailed standard designs have been provided and frequently used. Travelling centering and steam curing of concrete cast in place have also been developed. Many millions of square feet of factories have been covered with new types of roofs since 1945.

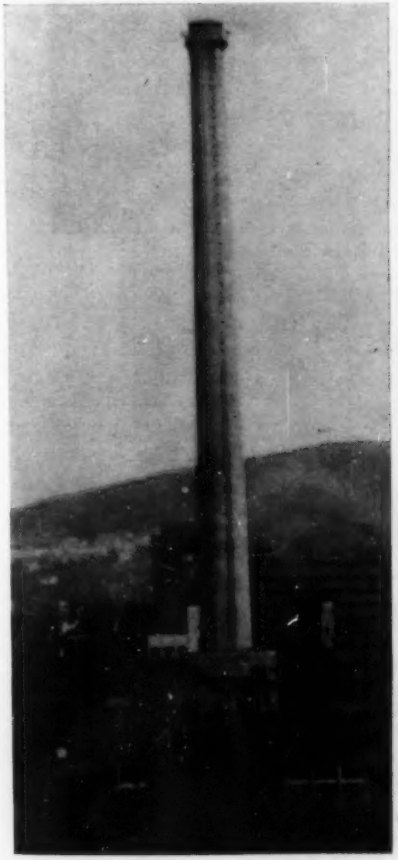


Fig. 9.—Chimney at Gottwaldov.

Fig. 10.—Chimney during Construction.

Fig. 11.—Part of an Exhibition Hall at Brno.

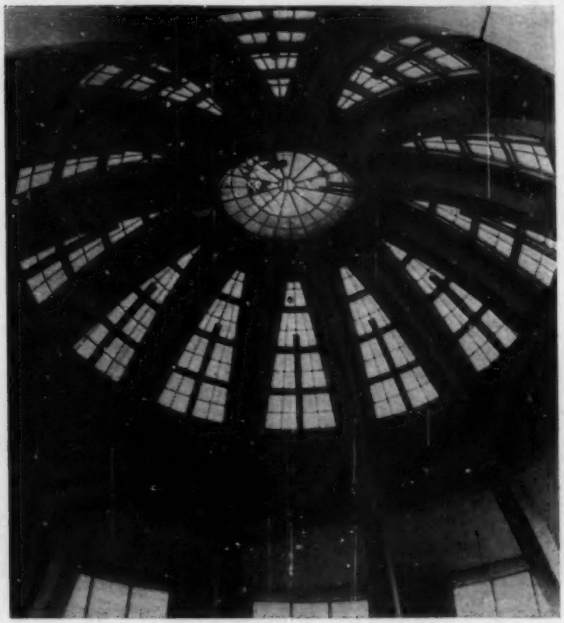


Fig. 12.—Roof at Exhibition Hall at Brno (built in 1928).

Fig. 13.—Cubicles at a Bathing Pool at Tr. Teplice.

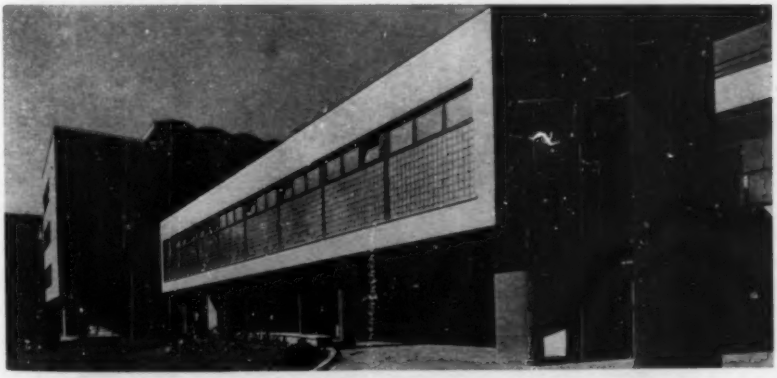


Fig. 14.—Part of a Hospital at Brno.

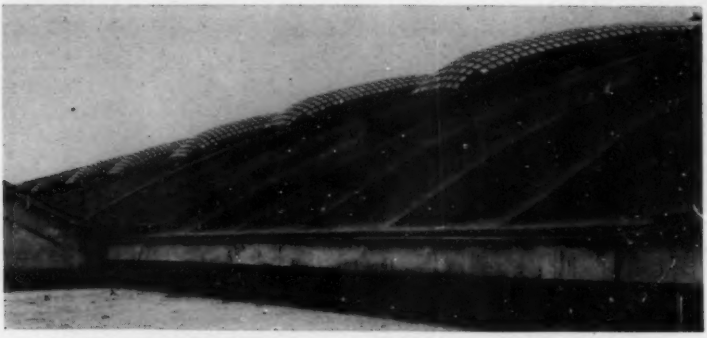


Fig. 15.—Hyperbolic-paraboloid Shell Roof, with Roof Lights.

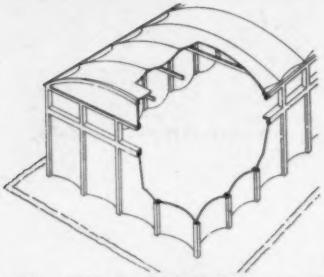


Fig. 16.—The Use of Thin Curved Slabs.

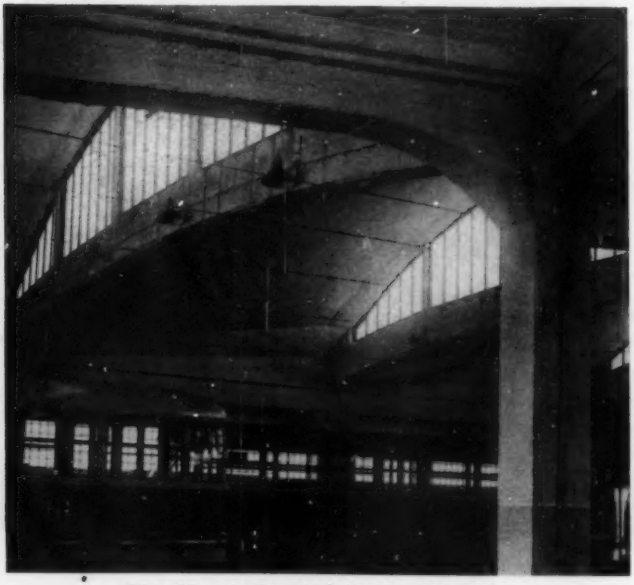


Fig. 17.—A Conoidal Roof at Prague.

" Shell " Roofs.

Experience has shown that if they are wisely applied and skilfully carried out shell roofs permit considerable reduction of both the weight and the cost of a structure. Some examples of the types of shell roofs now in use in Czechoslovakia are given in *Figs.* 15 to 21. The trough-shaped type of shell roof (*Figs.* 15 and 16) for spans of 50 ft. to 90 ft. was described in this journal for August, 1949, pages 247 to 251. *Fig.* 16 shows the design of a structure at Neratovice, 20 ft. high, for the storage of salt ; the filling between the columns and beams consists entirely of curved members.

Conoidal surfaces also have certain advantages in cases when the grid of columns is in the form of long rectangles (*Figs.* 17 and 18). The roof shown in

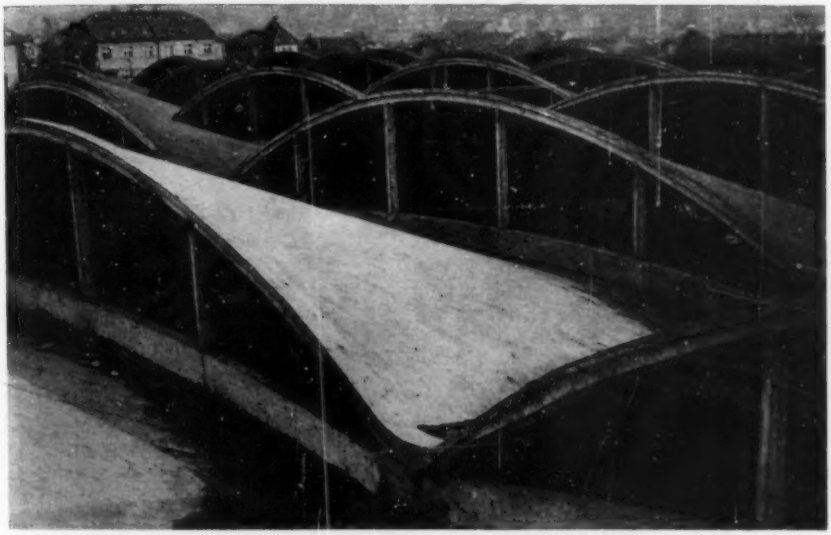


Fig. 18.—A Conoidal Roof during Construction.

Fig. 17 is over a tramcar depot in Prague, and comprises five bays of 59 ft. span. Where a uniform distribution of light over the whole floor area is required, saw-tooth roofs with long cylindrical slabs are used. Original designs of parabolic shapes have been built such as that shown in *Fig.* 19, where the spans of the bays are from 39 ft. to 59 ft. and the span of the arch ribs is 29 ft. 6 in.

The use of travelling centering combined with quick curing and hardening of the concrete make it possible to use economically even more complicated thin slab systems. For example, several industrial structures have roofs as shown in the cross section in *Fig.* 20. The arched beams are spaced 20 ft. apart, and between them the smooth white soffits of the parabolic slabs act as efficient light reflectors and dispersers. The slabs are 2 in. to 3 in. thick. With the use of precast columns a weekly rate of up to 6000 sq. ft. of completed roof has been achieved.

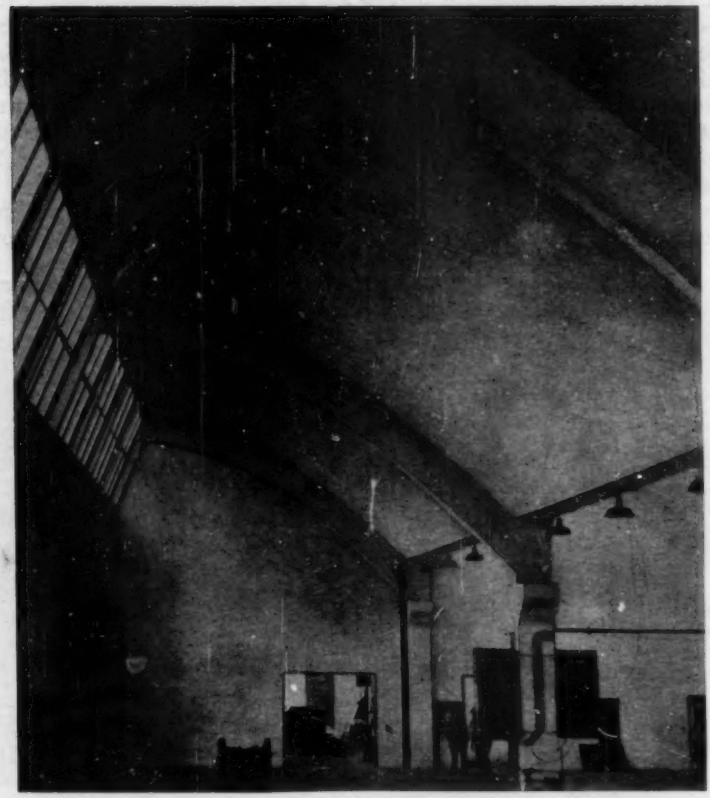


Fig. 19.—A Parabolic Roof.

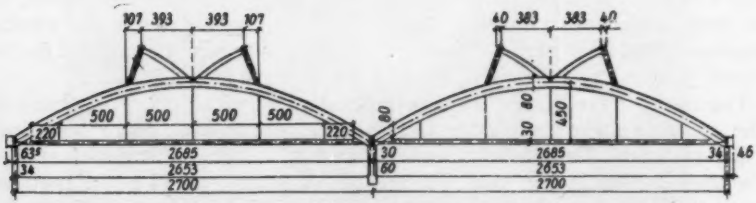


Fig. 20.—Shell Roof with Saw-tooth Lighting.

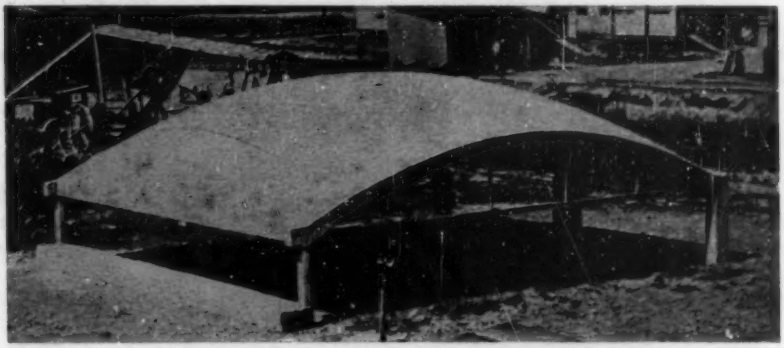


Fig. 21.—Model (one-third full size) for Testing.

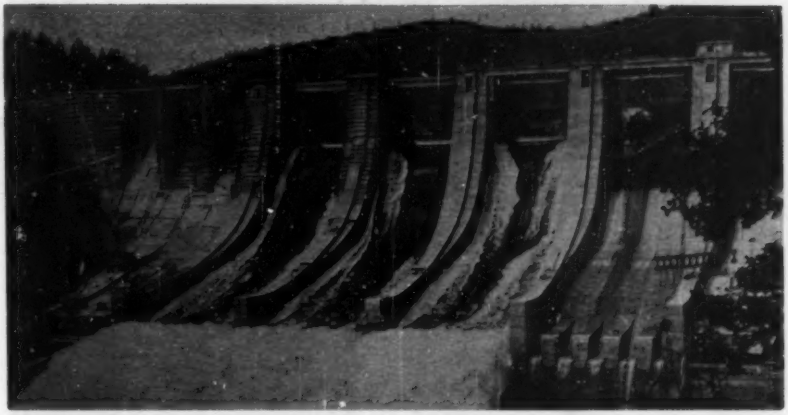


Fig. 22.—Dam at Vltava.

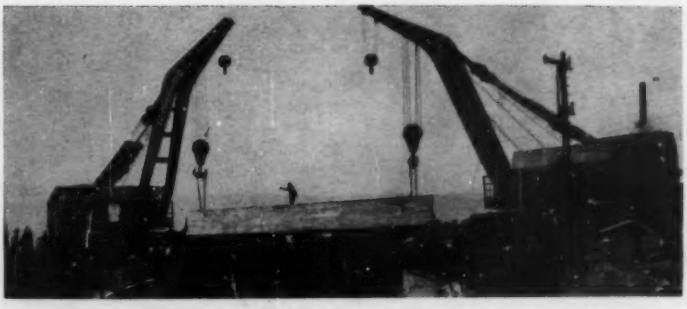


Fig. 23.—Erecting a Beam weighing 100 Tons.

The roof of another structure is composed of eight members of the shape shown in *Fig.* 21, spanning 59 ft. by 69 ft. The slab is $2\frac{3}{4}$ in. thick. Travelling centering and steam curing were used. All the members were cast in the same mould within eight weeks.

A reinforced concrete dam at Vltava is shown during construction in *Fig.* 22.

Precast and Prestressed Concrete.

The post-war necessity to increase the productivity of the building industry has resulted in the rapid development of precast and prestressed structural members. Many times more precast members were used in 1955 than in 1948, and a further rapid increase of production is planned.

At present most bridges with spans of less than 180 ft. are constructed of

Fig. 24.—Early Type of Prestressed Road Bridge.

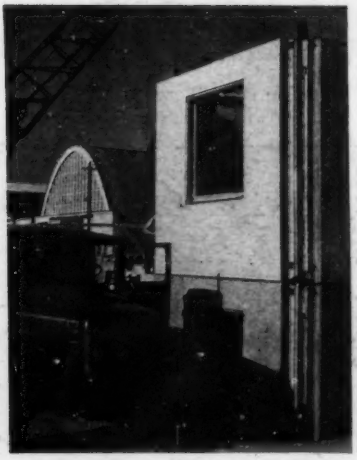


Fig. 25.—Erecting Precast Wall Slabs.

Fig. 26.—Erecting Precast Wall Slabs.

Fig. 27.—Residential Flats with Prestressed Precast Wall Slabs.

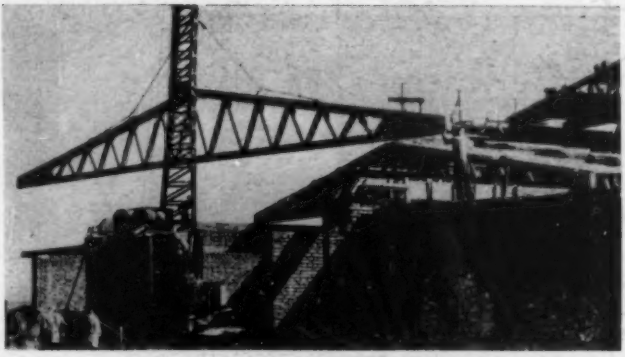


Fig. 28.—Erecting Roof Trusses.

Fig. 29.—Erecting Precast Roof Arches of 79 ft. Span.

precast elements (*Figs.* 23 and 24). The construction shown in *Fig.* 24, using I-beams prestressed by the post-tensioning process for spans up to 180 ft., is not now used. In the later types of prestressed structures it is common to use box-girders precast in sections and made statically indeterminate by tensioning additional cables after erection.

Three types of lightweight reinforced concrete construction have been developed for dwellings. The latest type is composed entirely of prestressed

Fig. 30.—Precast Roof of Store Building.

Fig. 31.—Erecting Precast Roof Member 8 ft. Wide.

Fig. 32.—Stores Built with Precast Members.

precast panels (*Fig.* 25) which are lifted into position by a new type of crane (*Fig.* 26). The panels are cast in steel moulds and are prestressed by a continuous process of pre-tensioning the steel on a rotating table. A completed block of flats is shown in *Fig.* 27.

Precast roof members for industrial structures are shown in *Figs.* 28 and 29. In some cases members comprising ribs and shells cast as one member have proved advantageous. The type shown in *Figs.* 30 to 32 is used for structures with floor areas up to 60 ft. by 200 ft.; eighteen such stores were built at Bratislava in 1954. The erection of one of those structures was completed in ten days. The amount of reinforcement used was 2·8 lb. per square foot of floor area. The roof shown in *Fig.* 30 has a span of 59 ft. and the slab is 1·2 in. thick. *Fig.* 32 shows how the roof members were stored in front of the buildings already erected.

A recent store building with an area of about 73,000 sq. ft. (*Figs.* 33 and 34) and a span of 127 ft. consists entirely of prestressed precast members. Arched

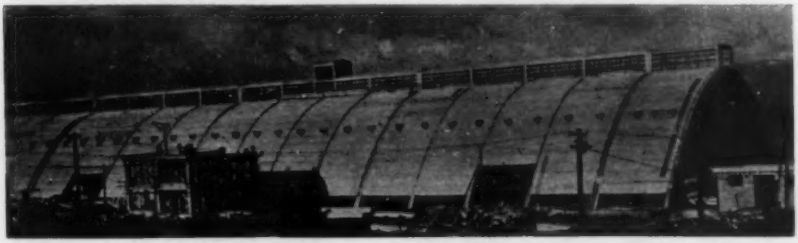


Fig. 33.—Store of 127 ft. Span built of Precast Members.

Fig. 34.—Erecting Precast Arches.

Fig. 35.—Device for Forming Ducts for Prestressing Steel.

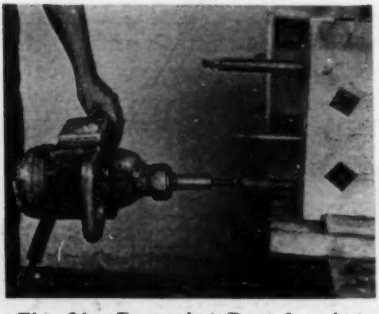


Fig. 36.—Removing Duct-forming Device.

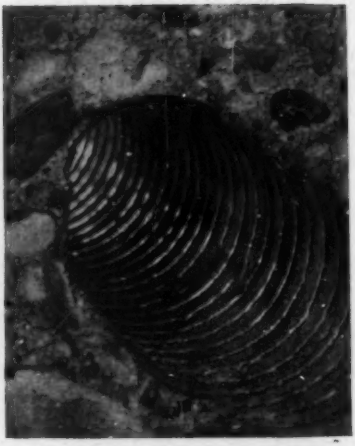


Fig. 37.—Corrugated Duct for Prestressing Steel.

trusses spaced 30 ft. apart are composed of several precast portions joined by post-tensioned prestressing cables. After erection the temporary hinges at the crown and at the springings were fixed by additional cables, so that the arch is rigidly fixed. The quantities of steel used were $\frac{1}{2}$ lb. of high-tensile wire and 3·6 lb. of mild steel per square foot of floor area.

Some new processes have been devised for use in prestressed concrete. For providing cable channels, for example, special insertions (*Fig.* 35) are used, consisting of a tube made of helically wound hard steel wire with a mild steel bar of $\frac{3}{4}$ in. diameter inserted loosely into it. When the concrete has hardened the tube and bar are easily removed by means of a drill (*Fig.* 36); this is possible even if the duct is curved. The helical groove left in the surface of the duct is shown in *Fig.* 37. After placing and tensioning the cables the duct is filled with cement mortar. By this means the cost of metal sheaths is saved and a reliable bond is provided for the grout.

Theory and Research.

Since 1948 all reinforced concrete structures except bridges have been designed according to their strength at failure,† that is by the ultimate-load method. This has resulted in saving about 12 per cent. of the amount of steel used in reinforced concrete by former rules based on permissible stresses.

At present research is concerned mainly with the ultimate stress of steel reinforcement (which is actually not the same as the yield-point stress of plain steel bars when tested in the usual manner), with statistical characteristics of the strength of reinforced concrete structures, and with various problems of stability.

† The writer described the method in this journal for December, 1949.

Reservoir at Cambridge.

UNUSUAL DESIGN OF WALLS.

THE new service reservoir of 5,000,000 gallons capacity (*Fig.* 1) constructed for the Cambridge Waterworks Company at Cherry Hinton is entirely of reinforced concrete. Alternative designs were considered using prestressed concrete and plain concrete but reinforced concrete was found to be considerably cheaper. The reservoir is 215 ft. square internally and the average depth from the floor to top water level is 18 ft.

Further reduction of stress due to changes of temperature is achieved by providing two expansion joints completely through the roof, walls, and floor in two directions at right angles, thus dividing the structure into nine separate parts. These joints are made with rubber-bitumen fillers, with continuous dumb-bell section rubber waterstops as a second line of defence.

Fig. 1.

The roof is of flat-slab construction supported on columns with splayed heads, thus reducing the height of the columns and walls and hence the cost. Other advantages of the flat-slab construction are that efficient ventilation is more easily effected, and fewer arrises occur that might lead to the rusting of the reinforcement and spalling of the concrete.

In order to reduce the stresses in the structure and cracks due to variations of temperature and loading, the roof is supported independently of the walls with a gap of $1\frac{1}{2}$ in. between the roof and the walls; this gap is filled with a rubber-bitumen sealer. This design also had the advantage of allowing the construction of the walls and roof to proceed independently, which enabled the whole of the concrete work to be completed in ten months.

When the reservoir was first filled with water at 45 deg. Fahr. in August, 1955, the roof moved inwards relatively to the wall by $\frac{1}{8}$ in. The force necessary to resist such a movement all around the

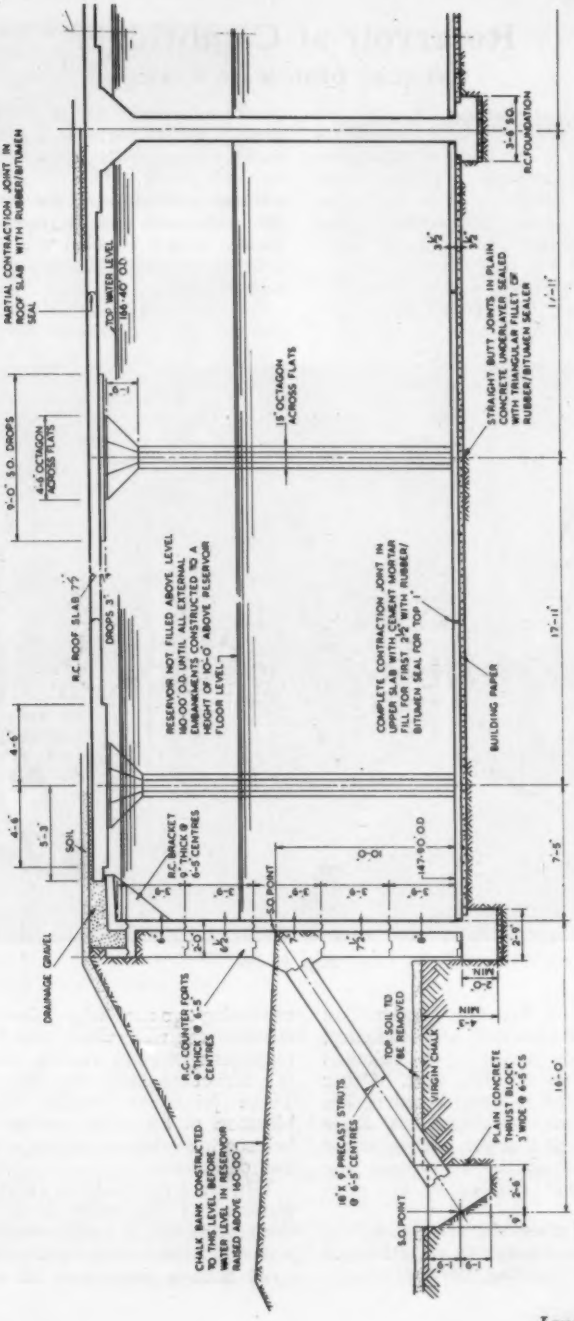

PARTIAL CONTRACTION JOINT IN ROOF SLAB WITH RUBBER/BITUMEN SEAL
TOP WATER LEVEL 166·40 O.D.
9'-0" S.O. DROPS
4'-6" OCTAGON ACROSS FLATS
15' OCTAGON ACROSS FLATS
R.C. ROOF SLAB 7½"
DROPS 3"
RESERVOIR NOT FILLED ABOVE LEVEL 160·00 O.D. UNTIL ALL EXTERNAL EMBANKMENTS CONSTRUCTED TO A HEIGHT OF 10'-0" ABOVE RESERVOIR FLOOR LEVEL
COMPLETE CONTRACTION JOINT IN UPPER SLAB WITH CEMENT MORTAR FILL FOR FIRST 2½" WITH RUBBER/BITUMEN SEAL FOR TOP 1"
STRAIGHT BUTT JOINTS IN PLAIN CONCRETE UNDERLAYER SEALED WITH TRIANGULAR FILLET OF RUBBER/BITUMEN SEALER
BUILDING PAPER
R.C. FOUNDATION
R.C. BRACKET 9" THICK @ 6'-5" CENTRES
S.O. POINT
147·80 O.D.
SOIL
DRAINAGE GRAVEL
R.C. COUNTER FORTS 9" THICK @ 6'-5" CENTRES
CHALK BANK CONSTRUCTED TO THIS LEVEL BEFORE WATER LEVEL IN RESERVOIR RAISED ABOVE 160·00
TOP SOIL TO BE REMOVED
VIRGIN CHALK
18" X 9" PRECAST STRUTS @ 6'-5" CENTRES
PLAIN CONCRETE THRUST BLOCK 3 WIDE @ 6'-5" CS
MIN
16'-0"
17'-11"
7'-5"

Fig. 2.—Part Cross Section.

reservoir acting against the total cross-sectional area of the whole roof slab could not practically have been resisted by the thin cross sections used, namely 7½ in. for the roof slab and 6 in. for the walls.

The walls span between counterforts 2 ft. by 9 in. in cross section, generally at 6 ft. 5 in. centres (*Figs.* 2 and 3). Advantage was taken of the width of the counterforts to design the walls for a bending moment of $\frac{wl^2}{18}$. The reinforcement is the same on both sides of the walls to resist the pressure of the external embankments. The counterforts are propped at mid-height by struts inclined at an angle of 35 deg. to the horizontal.

In determining the spacing of the columns and the counterforts, the cantilever moments at expansion joints were balanced with the support moments arising from the loads on adjacent spans; nevertheless it was possible to arrange for the expansion joints in the walls and roof to coincide.

A feature that led to considerable economy was the simple arrangement of the reinforcement (*Figs.* 4 and 5). All the bars in the roof are of the same size, with generally only two shapes of bending. In the walls, all the bars are straight and of only two diameters, and in the floor all the bars are straight and of the same diameter.

The amount of concrete placed in any

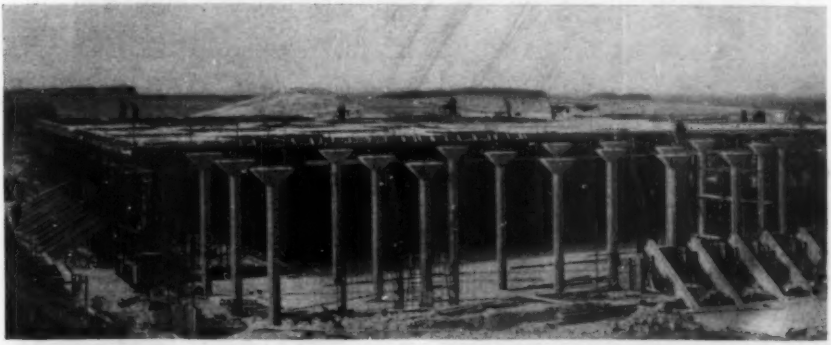


Fig. 3.

The struts were precast and placed in position before the thrust-blocks and counterforts were concreted.

The floor is in two layers both 3½ in. thick. The lower layer is unreinforced and the upper layer is reinforced with a mesh of bars placed centrally in the slab at 10-in. intervals. The top surface of the lower layer was rubbed with a steel float to a smooth finish simulating granolithic and covered with building paper before the upper slab was concreted. Both slabs were concreted in panels 17 ft. 11 in. square to coincide with the spacing of the columns with straight joints staggered between the two slabs. A gap was provided to allow complete freedom at the columns, and all the joints in both slabs were sealed with a rubber-bitumen compound.

one operation was strictly controlled. In the case of the walls this was limited to 19 ft. 3 in. horizontally by 3 ft. 9 in. vertically; in the roof the limit was generally 17 ft. 11 in. in both directions. An interval of fourteen days was allowed before the intermediate panels were cast in order to allow shrinkage to take place in the panels cast first. The partial contraction joints necessitated by this procedure were pointed with rubber-bitumen sealers.

To prevent leakage by the water bypassing the rubber-bitumen seals, the internal face of the concrete was painted with a bituminous primer followed by two coats of bituminous paint (*Fig.* 6).

Before starting the work, test cubes and slabs were made. The slabs were 1 ft. square and 4 in. thick and were

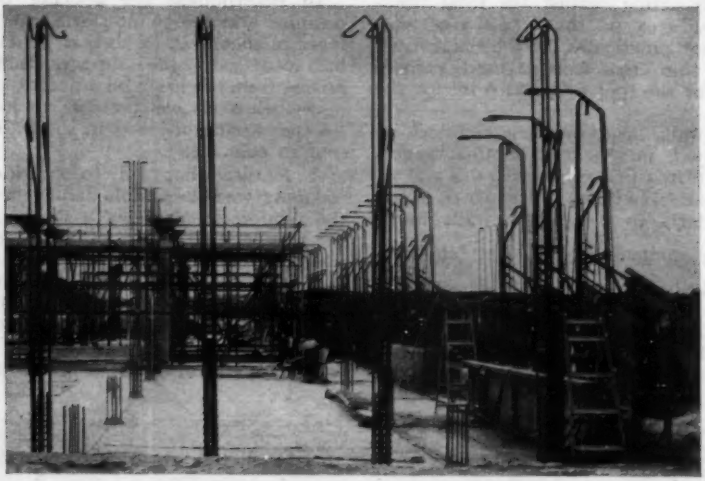


Fig. 4.—Reinforcement for Walls.

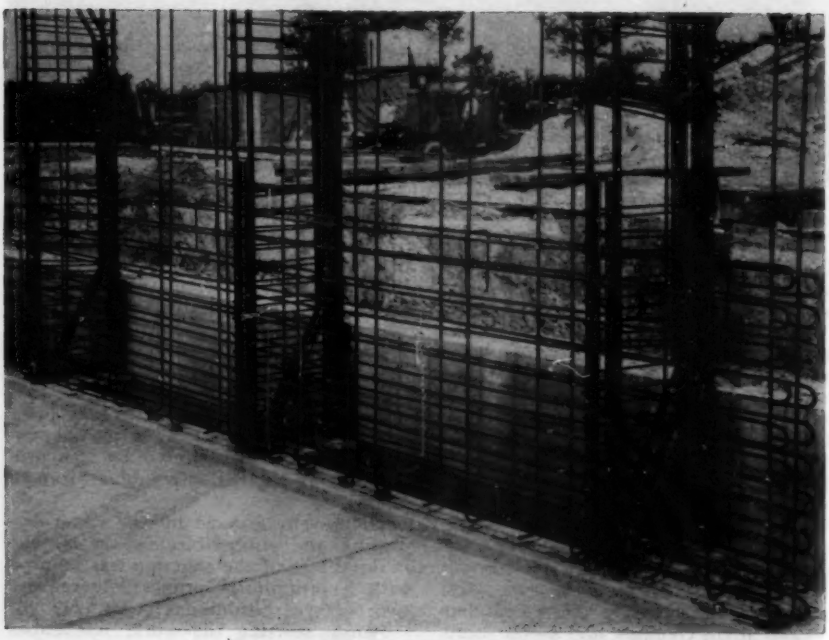


Fig. 5.—Reinforcement for Wall.

subjected when fourteen days old to a 40-ft. head of water for four days; after this period the sides of the slabs remote from the water pressure were completely dry. Test cubes and slabs were made and tested in a laboratory at intervals throughout the work.

All concrete was compacted by vibration, and in addition the concrete in the walls and columns was agitated by sword-type spatulas 6 ft. long and 2 in. by ¾ in. in cross section, which were thrust into the concrete between the reinforcement and the shuttering. The shuttering to vertical surfaces was struck about four hours after placing the concrete, and the concrete immediately rubbed with a steel trowel to a uniformly smooth surface. This improved the appearance, and is believed to have contributed to the complete water-tightness of the reservoir.

The main work of constructing the reservoir cost about £60,000 and the reservoir was put into commission in August, 1955. In the course of excavation at one end of the reservoir an old quarry working 20 ft. deep was exposed; measures to deal with this cost a further £7000 or so.

At the end of the maintenance period in 1956 readings were made of the water level of the reservoir when full, using a hook-gauge that could be read to about one-fiftieth of an inch, and during a period of one week no fall in the water level was registered.

The Engineer and Managing Director of the Cambridge Waterworks Company is Mr. Philip Porteous, M.I.C.E., M.I.W.E. The contractors for the work were Messrs. William Sindall, Ltd., of Cambridge, and Mr. A. Durrant, B.Sc., A.M.I.C.E., acted as resident engineer. The consulting engineers were Messrs. Oscar Faber & Partners.

Fig. 6.—Inner Face of Wall.

Aggregates for Concrete Roads.

ACCORDING to the Report of the Road Research Board for the year 1955, an experimental concrete road built in 1933 shows that the use of crushed rock aggregate instead of a cheaper local stone has resulted in improved wearing properties equal to about 1 in. of the thickness of the slab.

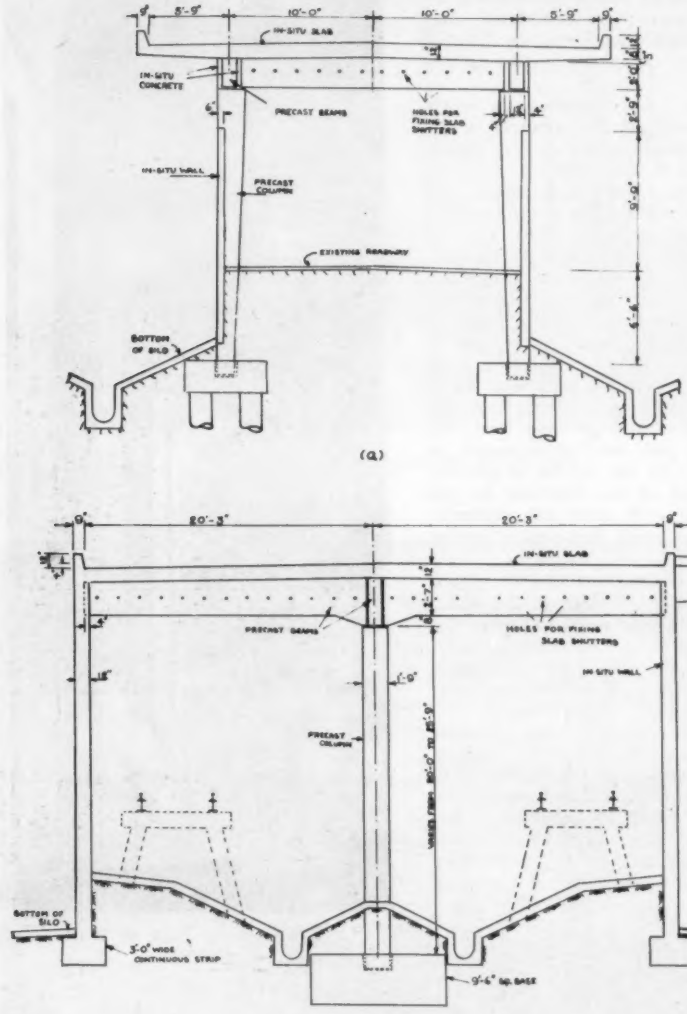


Fig. 3.—Cross Sections through Unloading Platforms.

(*See facing page.*)

Overhead Roads at Sugar Factories.

Both the overhead roads described in the following were built for the British Sugar Corporation, Ltd., one at Poppleton, near York (*Figs.* 1 and 2), and the other at Peterborough. The designs were varied only to suit local conditions.

The structures comprise (1) main unloading platform more than 20 ft. above the bottom of the adjoining silos, (2) entrance ramp with a gradient of 1 in 12, and (3) an exit ramp with a gradient of 1 in 8¼. The dimensions of the main platform were determined by local conditions and it is sufficiently wide to allow lorries to be easily manœuvred. At Peterborough the platform is 380 ft. long by 41 ft. 6 in. between the kerbs, and at Poppleton it is 360 ft. by 31 ft. 6 in. A kerb 1 ft. high is provided at both edges. There are no balustrades, as these would interfere with unloading; there is, however, a safety grillage projecting beyond the edge as seen in *Fig.* 2. As the traffic is in one direction only a width of 12 ft. was considered sufficient for the entrance and exit ramps. The entrance ramp is curved in both cases, the internal radius being 35 ft.

The structures were designed for vehicles weighing 20 tons. The walls and frames under the main unloading platform had also to resist a horizontal thrust from the beet in the adjoining silos. At Poppleton these walls also retain the road under the main platform (*Fig.* 4). A typical section through the main unloading platform at Poppleton is shown in *Fig.* 3 (a) (see facing page) and the arrangement at Peterborough is shown at (b). At Peterborough the retaining walls, 1 ft. thick, extend up to the unloading platform and are used as external supports for the slab. There are columns, 1 ft. 9 in. by 1 ft. in cross section at 20 ft. centres, under the centre-line of the road. Longitudinal beams and cross beams, 3 ft. 7 in. deep by 1 ft. wide and haunched at the

Fig. 1.—Underside of Entrance Ramp.

Fig. 2.—Overhead Road at Poppleton.

internal supports, divide the road slab, which is 1 ft. thick, into panels 20 ft. square spanning in two directions.

At Poppleton the 6-in. retaining walls do not extend the full height of the structure, and span between tapered columns at 20 ft. centres. Longitudinal and cross beams 3 ft. deep by 1 ft. wide support the road slab, forming panels 20 ft. square that extend 6 ft. as cantilevers on both sides. A typical section through the curved entrance ramps to both roads is shown in *Fig.* 5 (a). The columns are 1 ft. square and are at 24 ft. centres at Poppleton and 20 ft. centres at Peterborough. A section through the exit ramp at Peterborough is shown in *Fig.* 5 (b). The exit ramp at Poppleton

Fig. 4.—Exit Ramp at Poppleton.

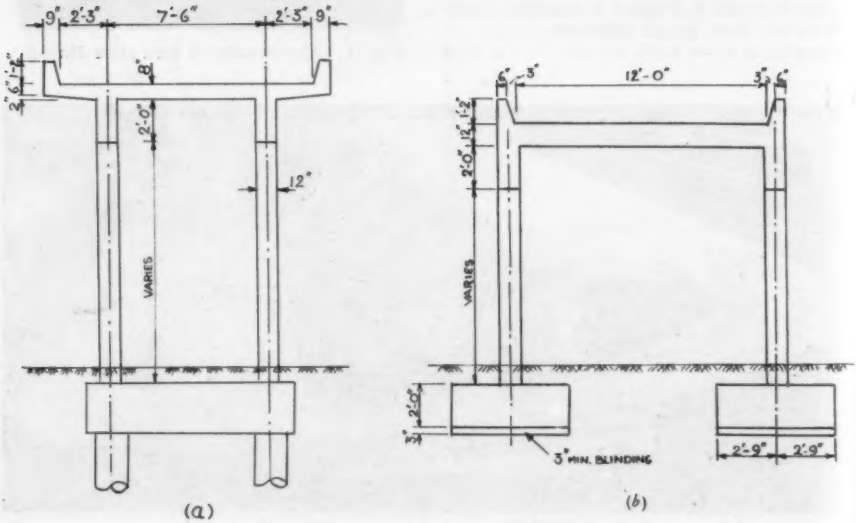


Fig. 5.—Cross Sections through Ramps.

(*Fig.* 4) is supported on a single row of columns measuring 2 ft. 3 in. by 1 ft. 3 in. and at 24 ft. centres.

The structure at Poppleton and the entrance ramp at Peterborough are on cast-in-place piles. The columns of the main unloading platform and exit ramp at Peterborough are supported on independent bases and the walls on continuous strip footings. Expansion joints in the slabs and the walls are provided at intervals of 50 ft. to 70 ft., and are arranged so as to avoid the necessity of constructing twin columns and beams.

In each case all the work had to be completed in about six months. The work on the site was reduced as much as possible. All the columns and beams for the main unloading platform were precast. Details of the connection of the precast units at Peterborough are shown in *Fig.* 6. The limited space at the junction of the four precast beams and the column required elaborate detailing. The 1-in. and $\frac{7}{8}$-in. bars projecting above the precast beams were used as lifting hooks. The continuity reinforcement in the top of the beams was fixed in situ.

Shutters made of timber and plywood were used for the construction of the curved entrance ramp at Poppleton (*Fig.* 8), and these were used later for the similar ramp at Peterborough.

The walls, the slab, and both the ramps were cast in place. Holes were provided in the precast beams (*Fig.* 3) for fixing

Fig. 6.—Connection of Column and Four Beams.

Fig. 7.—Erecting Precast Beams.

Fig. 8.—Shutters for Curved Entrance Ramps.

the shuttering for the roadway slab, so that little propping and strutting were required. *Fig.* 7 shows the erection of precast beams at Poppleton. At Peterborough a truck on the railway lines under the overhead roadway was used to support the shuttering for the walls (*Fig.* 9).

On both sites the work was started at the exit ramp, which was then used for transporting concrete in dumpers to the main road slab.

The design was carried out by Messrs. Hajnal & Myers, consulting engineers, for Messrs. Charles R. Price, the main contractors.

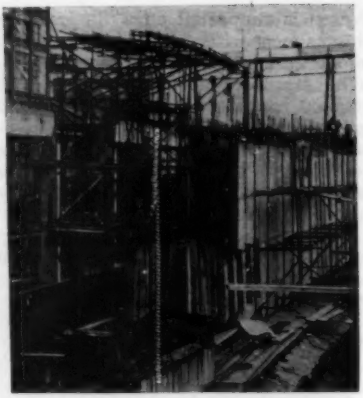


Fig. 9.—Wall Shutters supported by Railway Truck.

Gantry at a Colliery.

THE gantry shown in *Fig.* 1 is a composite reinforced and prestressed concrete structure. It is 900 ft. long, and rises from ground level to a height of 50 ft. at a gradient of 1 in 17. The conveyor is 7 ft. square internally. The trestles were precast and the capping beams cast in place.

The longitudinal beams were prestressed with post-tensioned bars. They are of tee-section, 2 ft. 1 in. deep and with top flanges 3 ft. 9 in. wide; they are 48 ft. 6 in. long, weigh 10 tons each, and are bolted to the tops of the trestles.

The posts, roof beams, and wall slabs of the conveyor are of precast prestressed concrete, and the roof is of curved asbestos cement sheets.

The structure has been built for the National Coal Board at Cadeby Colliery by the Yorkshire Hennebique Contracting Co., Ltd.

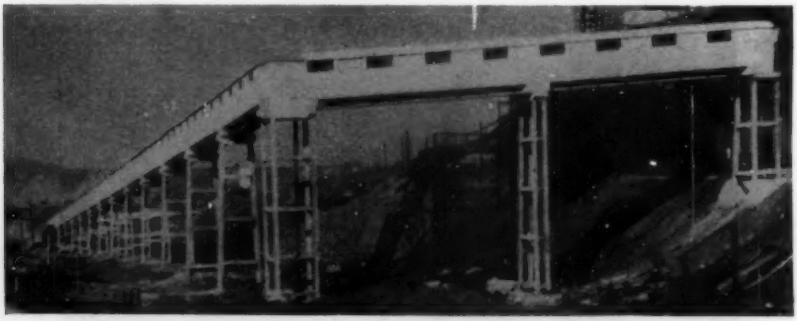


Fig. 1.

Strip Mill and Bridge at Rotherham.

THE new strip mill being built at Rotherham for Messrs. Steel Peech & Tozer, at an estimated cost of £3,600,000, is now well advanced. The civil engineering work is completed and the installation of the mill is in progress. The site is on open land near the main works of the Company, which produces over 1,000,000 tons of steel per annum and includes a strip mill. The new mill will produce strip up to 16 in. wide of many qualities and thicknesses, including stainless steel.

To provide access to the railway a new connection, about half a mile long, with the works railways was made, and as the site of the new mill extends into the side of a hill a maximum depth of excavation of about 42 ft. was necessary. The total amount of material excavated exceeded 300,000 cu. yd., a large part of which was carboniferous shale and other soft rock, and more than two-thirds of it had to be removed from the site.

At the lower end of the site the soil had a very small bearing value and the foundations of a large part of the mill, which is in this area, were taken down to rock; the adjacent welfare and office building was supported on piles. In the adjacent bay, owing to the extremely high loading from the steel slabs, trenches were excavated to rock at close intervals and filled with concrete.

The main strip mill building is of structural steel weighing 3400 tons. It is 800 ft. long and 500 ft. wide and over a substantial part of the motor bay and the mill bay there is a basement to accommodate the transformer, oil, and scale-water flumes. The adjacent pickling plant is also founded on rock, which occurs near the surface. Adjacent to the western end of the mill is the recuperator house and the chimney serving the slab re-heating furnaces, which are at the western end of the mill building. Nearby is the scale-water pit, which is 80 ft. by 20 ft. by 34 ft. deep, with a crane gantry overhead for the removal of the sludge. Alongside the scale-water pit is the pump-house to deal with the recovered scale-water. The welfare and office building, now under construction, is immediately in front facing the main Sheffield to Rotherham road.

The greatest amount of the concrete work is in the foundations and basement of the mill building, and most of this is reinforced concrete in the proportions of 1 : 2 : 4 with coarse aggregate of $\frac{7}{8}$ in. maximum size and ordinary Portland cement; the water-cement ratio was low to reduce shrinkage stresses in the large masses of concrete cast in one operation in some parts of the work. Elsewhere, concreting was generally carried out in alternate sections, mainly with an interval of five days between the concreting of adjacent bays. *Fig.* 1 is a typical cross section through a part of the substructure.

The Bridge.

A bridge was required where the approach railway cutting is about 24 ft. deep and an existing road crossed the site, and a little farther along there were also several water mains and electric cables. To avoid interrupting the use of the road and of these services, the bridge (*Fig.* 2) was built before the excavation for the railway cutting was completed. The water mains and cables were first re-laid on temporary concrete pads in the position to be occupied by the bridge and slightly below its deck, so that they would

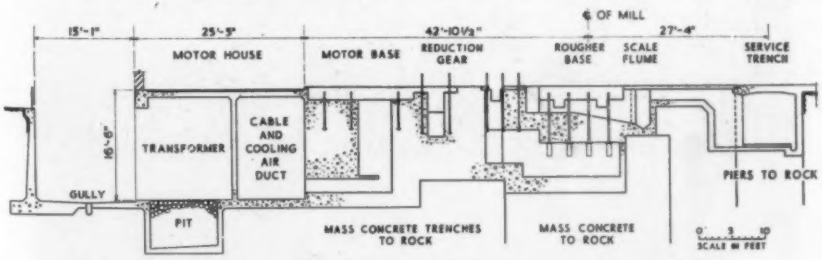
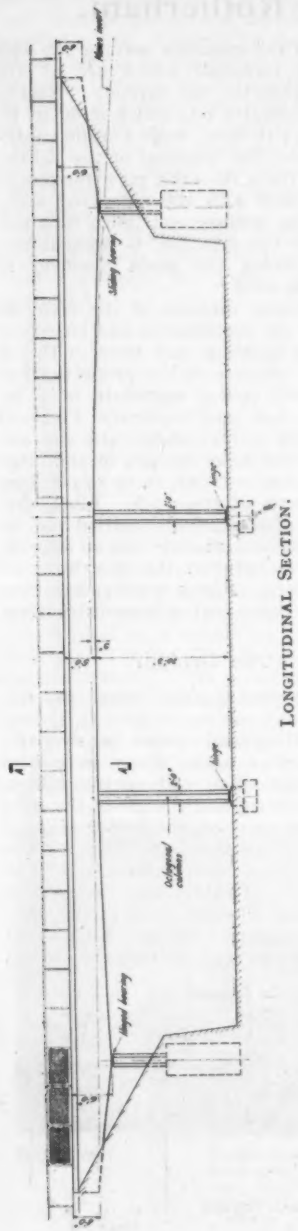
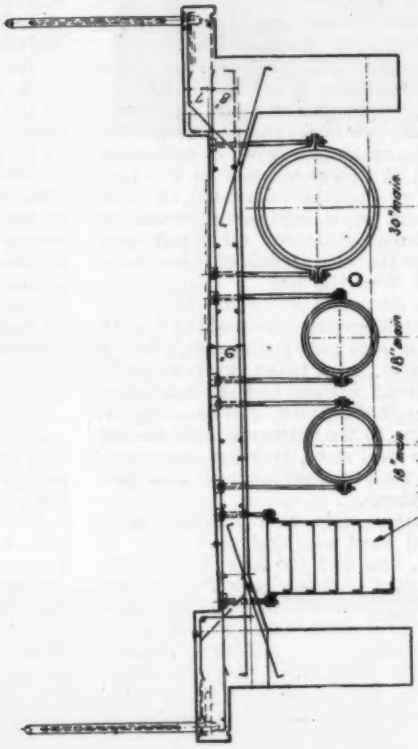


Fig. 1.—Typical Section through Substructure of Mill Building.



LONGITUDINAL SECTION.



CROSS SECTION.

Fig. 2.—Bridge over Railway Cutting.

Figs. 3a and 3b.—Water Mains and Cables Suspended while Foundations of Bridge are Built.

be subsequently supported by the bridge above the railway tracks 24 ft. below. The piers were then constructed in excavations, and after the deck was concreted the weight of the water mains and the electric cables was transferred to the bridge, precautions being taken not to change the levels of the water mains. When this was completed the excavation for the railway cutting was carried under the bridge. The bridge has a total length of 170 ft., including spans of 40 ft., 43 ft., and 47 ft., and cantilevers extending 20 ft. beyond the end piers. The overall width is 17 ft., comprising a roadway 11 ft. wide and footpaths on each side.

Figs. 3a and *3b* show the bridge under construction, and *Fig. 4* shows the bridge and the excavation underneath completed and the railway tracks being laid. The concrete in the bridge is a 1 : 2 : 4 mixture with a slump not exceeding 1 in. and was compacted by vibration. High-grade concrete is considered particularly desirable in this area owing to the acid-polluted atmosphere at certain times. For the same reason the handrailing is of mild steel tubing shot-blasted and sprayed with pure aluminium ; the handrail is also designed to be removable in sections in case of damage by vehicles.

The whole of the concrete for the scale pit and the adjacent filter house and pump house was placed by concrete pump, and this method was also used for parts of the sub-structure of the mill building.

Mr. V. Watkins, Chief Engineer of Messrs. Steel Peech & Tozer, is responsible for the project, with the assistance of Mr. J. Waller and Mr. F. Parkin. Mr. Donovan Lee, M.I.C.E., is the consultant for the civil engineering work. The United Steel Structural Co. are respon-

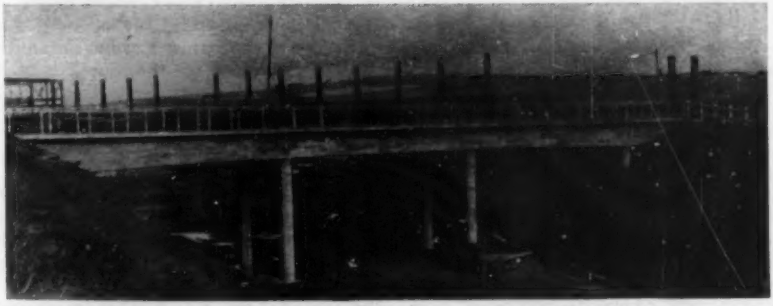


Fig. 4.

Fig. 2.—Bridge over Railway Cutting.

sible for the steel frame and roof. The contractors for the civil engineering work include Sir Alfred McAlpine & Son, Ltd., Messrs. George Longden & Son, Ltd., Messrs. Arthur Cooper & Sons, Ltd., and Messrs. Henry Boot & Sons, Ltd. The conversion of the mains was carried out by Messrs. E. E. Jeavons & Co., Ltd., the railways by Industrial Sidings, Ltd., and the furnaces by Messrs. Stein & Atkinson. The Loewy Engineering Co., Ltd., and the English Electric Co., Ltd., are responsible for the mill and the main electrical installation.

The Late Peter Lind.

It is with much regret that we record the death, which occurred on December 1, 1956, after a short illness, of Herman Peter Thygesen Lind, the founder and chairman of Messrs. Peter Lind & Co., Ltd.

Born in Denmark in 1890, Peter Lind was a man of unusual ability, energy, and courage. At the age of twenty-five he had acquired a sound knowledge of reinforced concrete design and construction and of the English language and, in partnership with the late A. Ingerslev, in 1915 started in business in London as a reinforced concrete designer and contractor. This was only fifteen years or so since reinforced concrete had been introduced into Great Britain and there was still prejudice against the use of the new material. With a staff of competent engineers, both British and Danish, the new company quickly became one of the most important in the industry, and did much to promote the use of reinforced concrete at a time when most such structures were designed on the proprietary systems introduced into this country earlier in the century by Mouchel, Considère, Coignet, and others. The company was responsible for a great number of industrial structures of all types, but perhaps its most notable achievement was the construction of the new Waterloo Bridge across the Thames. During the first world war the firm built reinforced concrete ships, and in the second world war took part in the construction of reinforced concrete caissons for Mulberry Harbour.

In 1936 the firm of Tileman & Co, Ltd., which specialised in the construction of cement works and chimneys throughout the world, became associated with the Lind group of companies under his chairmanship. Lind was also chairman of a number of aggregate-producing companies in this country and in Canada, and a director of Scandinavian Airlines System, Ltd. He was a Member of the Institution of Civil Engineers and a Member of the Council of the Federation of Civil Engineering Contractors.

Lind took much interest in Danish affairs in London, and particularly in the Danish Church in London. He was a past president of the Danish Club in London, a trustee of the Danish Church Committee, and a member of the council of the Scandinavian Benevolent Society. He was a Knight Commander of the Order of Dannebrog.

The comparative youth of the reinforced concrete industry is shown in a striking manner when it can be said that a man who passed away last month at the age of 66 was one of its pioneers, for Peter Lind was indeed a pioneer whose competence as an engineer and as a man of business did much to further its interests.

A Large Precast Warehouse.

PRECAST FRAMES OF 88 FT. SPAN.

A WAREHOUSE (*Fig.* 1) for the storage of spare parts now being built at Doncaster for the International Harvester Co. of Gt. Britain, Ltd., incorporates reinforced concrete precast frames of unusual spans and spacing. The overall size of the building is 352 ft. by 220 ft., and within this area there are only sixteen internal columns. The spacing of the columns is related to the positions of storage bins and other fixtures that will be required.

Across the width of the building the roof, which comprises asbestos-cement sheeting with insulation-board underlining, is carried by five main frames, four of 66 ft. span and one of 88 ft. span. These frames are at 44 ft. centres. In order to shorten the period of erection and to reduce the weight of the frames, intermediate rafters were introduced at 14 ft. 8 in. centres. The intermediate rafters are carried on reinforced concrete precast valley beams 44 ft. long, which

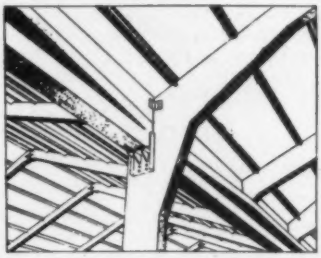


Fig. 3.—Detail of Joint.

span longitudinally between main frames. The valley beams weigh about 16 tons, and are carried on special brackets bolted to the columns of the main frames. The frames and valley beams were all precast on the site.

Fig. 2 is a half cross section. Scarfed, in-situ, and hinged joints have all been employed. Scarfed joints were used for

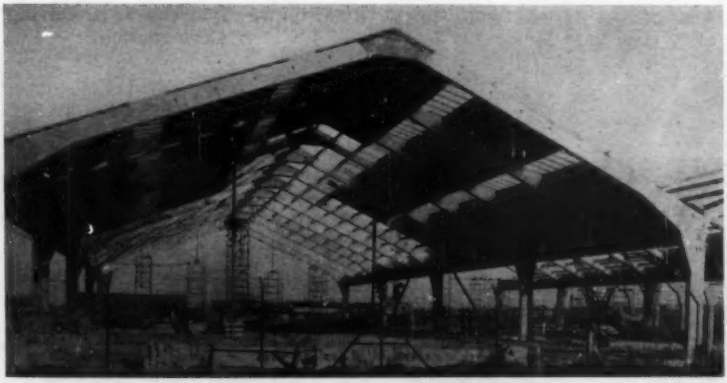


Fig. 1.—A Bay During Erection.

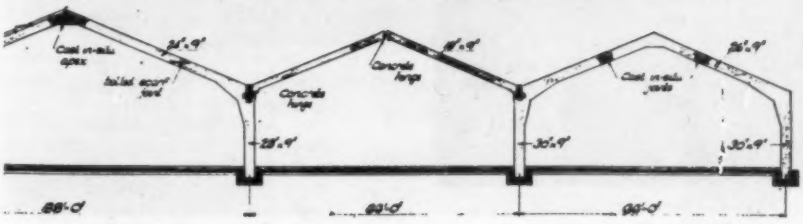


Fig. 2.—Half Cross Section.

ease of erection, in-situ joints for continuity, and hinged joints to reduce expansion and contraction and to enable the behaviour of the structure to be predicted as accurately as possible. The joint between a main frame and a valley beam is shown in *Fig.* 3.

Fig. 1 shows four 44-ft. bays of the central 88-ft. span completed and partly roofed and glazed. The architects are Messrs. T. H. Johnson & Son, of Doncaster, and the structure was designed, manufactured, and erected by the Scottish Construction Co., Ltd.

Gandy Bridge, Florida, U.S.A.

GANDY bridge (*Fig.* 1) at the head of Tampa Bay, Florida, U.S.A. has been built beside an existing bridge, each of which will carry one-way traffic. The new structure is 2½ miles long, and at the channel used by ships has a clearance of 45 ft. There are 252 spans of 48 ft., 20 spans of 72 ft., and one continuous span over piers spaced 74 ft., 86 ft., and 74 ft. apart. The design is generally similar to that of the lower Tampa Bay bridge described in this journal for February, 1953, but the beams are slightly wider; four beams instead of six are used for each span. The deck is of reinforced concrete cast in place, and the precast beams were prestressed on the Lee-McCall system using high-tensile alloy steel bars as for the lower Tampa Bay bridge but 1⅛ in. diameter instead of 1 in.

The concrete contained 585 lb. of cement per cubic yard; the coarse aggregate was crushed limestone, and the average compressive strength of cylinders at 28 days was usually about 5500 lb. per square inch. The high-tensile bars were tensioned to 100,000 lb. per square inch and tensioning was permitted when the strength of cylinders was 3600 lb. per square inch or the concrete was not less than five days old. The bridge is supported on reinforced concrete piles.

Fig. 1.—New Bridge (on right) **During Construction.**

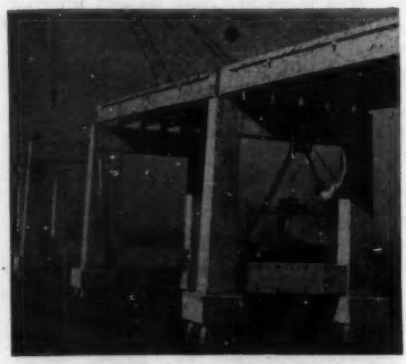


Fig. 2.—Spans of 72 ft. During Construction.

Fig. 3 shows the prestressed beams and diaphragms in the 48-ft. spans; *Fig.* 4 shows the erection of the prestressed beams in the 72-ft. spans, and in *Fig.* 2 are shown the supporting trestles and the deck-beams in the 72-ft. spans.

The total cost was $2,600,000, the cost of the 48-ft. spans was $4.75 (about 35*s.*) per square foot, and the 72-ft. spans, which were at a greater height, $11.86 (about 85*s.*) The Hardaway Contracting Co. were the contractors, and Mr. W. E. Dean, State Bridge Engineer of Florida, was responsible for the design and the supervision of the construction.

Fig. 3.—Prestressed Beams and Diaphragms in 48-ft. Spans.

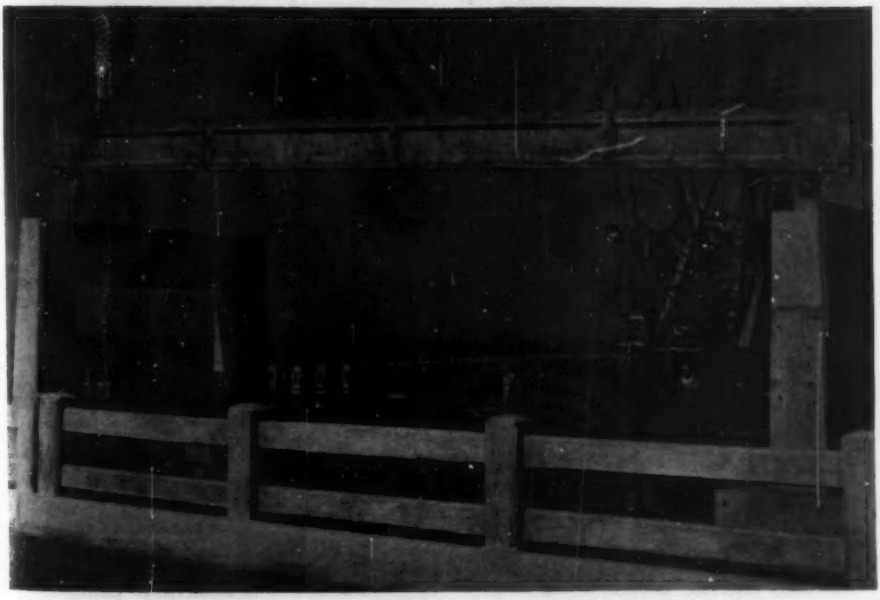


Fig. 4.—Placing Prestressed Girders for 72-ft. Spans.

The Strength of Concrete when Wet and Dry.

MR. LESLIE TURNER, B.Sc., M.I.C.E., Past President of the Institution of Structural Engineers, writes as follows.

In the last paragraph of the Editorial Note in your November, 1956, number it is stated that " we have the unexplained fact that a plain concrete beam subjected to a bending test may be stronger when wet than when dry ".

This phenomenon can, I think, be well accounted for by the facts and theory given in a report of some research on this question which I carried out some twenty-five years ago and which was published by the Institution of Civil Engineers in Selected Engineering Paper No. 165, 1934, " Plain and Reinforced Concrete in Torsion, With particular reference to Reinforced Concrete Beams ". Therein the true ultimate tensile stress was compared with the modulus found in bending. " Modulus " is a rather vague term, although still used in codes, and is the apparent ultimate stress of concrete when calculated by the elastic theory.

Now the deviation of concrete in tension only from the elastic, or straight-line, stress-strain relationship is very marked, particularly at ultimate stress, and the ultimate stress occurs at failure in most structures and certainly at failure by bending in beams. Thus the average stress-strain relationship found for 1 : 2 : 4 concrete (at a distance x from the tensile face towards the neutral axis) was $t_{ult} - K\frac{x}{2}3$, which results in a rectangular-beam formula of $MR = 0{\cdot}207tbd^2$, in which t is the true ultimate tensile strength.

The equivalent deviation in compression, due to decrease in ultimate compressive stress when wet, does not affect the equation as the stress-strain curve is sensibly straight up to about one-tenth of the ultimate stress in compression, while the alteration in the position of the neutral axis is insignificant.

This moment of resistance may be compared with the modulus calculated from the usual formula $MR = 0{\cdot}167tbd^2$ in the elastic theory, whence the apparent (or modulus) stress is 24 per cent. higher than the real tensile strength; or, conversely, the real tensile strength is only 0·8 of the modulus. It is this high plastic characteristic of concrete in tension that accounts for the disparity between the actual and apparent ultimate tensile stress, thus contributing to the difference between the actual and calculated loads at failure. It may be emphasised, however, that the two characteristics of high increase of strain at ultimate tension and increase of ultimate strength in tension when wet are of considerable importance and advantage when dealing with concrete subjected to wetting and saturation, such as water-retaining structures. Taking this a stage farther and comparing dry and wet beams (or any other members), the higher the true ultimate tensile strength the stronger the beam.

Reverting to your article, it is stated that " It is well known that hardened cement and concrete are stronger when they are in a dry state than when they are wet." Conversely, tests have also proved that concrete is stronger in tension when in a wet state than when dry, which was the reason all my specimens were, for the sake of consistency, tested in a saturated state.

If this statement be conceded and applied with the foregoing formula, it would appear that the fact that " a plain concrete beam subjected to a bending test may be stronger when wet than when dry " can be satisfactorily explained and confirmed. This would not exclude, in a future analysis, possible effects from other causes at present unknown or unappreciated, although it would be reasonable to assume that the explanation given in the foregoing accounts for the major influence.

Open-air Swimming Pools.

A BIBLIOGRAPHY of articles on swimming pools has been compiled by the Cement and Concrete Association (52 Grosvenor Gardens, London, S.W.1) from whom copies may be had free of charge. The articles appear in technical journals published throughout the world, and these may be seen on application to the Librarian of the Cement and Concrete Association.

Factory Extension at Croydon.

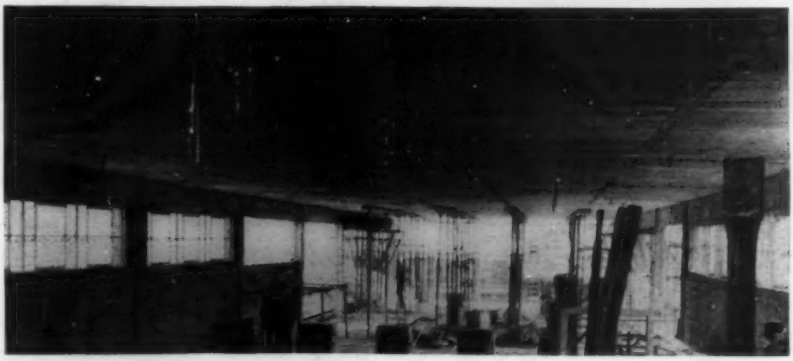


Fig. 1.—View of Lower Story.

A FEATURE of an extension to a factory at Croydon for Messrs. Bourjois, Ltd. (makers of perfume and toilet soap), was the requirement that there should be no beams projecting below the upper floor and the roof and that the positions of partitions should be unrestricted. The extension measures 188 ft. by 33 ft. and is two stories high.

The columns at the periphery are of reinforced concrete and the walls are of brick, capped by reinforced concrete beams. The upper floor and roof are of prestressed concrete on the Raphcon system (described in this journal for August, 1955). The floor was designed for an imposed load of 100 lb. per square foot plus 10 lb. for finishes, and the roof for 30 lb. per square foot plus 20 lb. for finishes.

The precast slabs were made with 1 : 1½ : 3 concrete and measure 6 ft. 1 in. by 1 ft. They are of hollow construction, 1 ft. 1½ in. thick in the floor and 10 in. thick in the roof. The ends of the slabs were supported by adjustable shores supporting boards that covered the gap of 4 in. to 5 in. left between the slabs ; these gaps were concreted to form beams in the longitudinal direction, and the slabs were then prestressed transversely by wires extending the full width of the span.

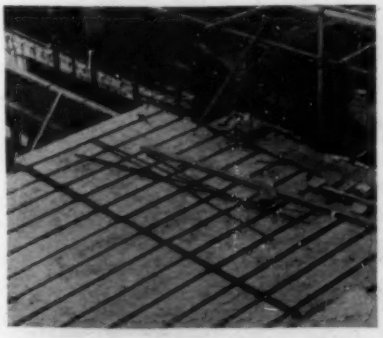


Fig. 2.—Precast Floor Slabs ready for Prestressing.

Fig. 3.—Edge-beam and Anchors for Wires.

The beams distribute the load and also reduce deflection. The prestressing wires, which generally comprised ten of 0·2 in. diameter in the floor and eight of the same diameter in the roof, are anchored to the edge-beams. The working stress in the concrete is about 1600 lb. per square inch. *Fig.* 1 shows the ground floor and the method of laying the slabs to form the longitudinal beams is shown in *Fig.* 2. *Fig.* 3 shows the anchorage at an edge-beam of the transverse prestressing wires.

The surveyors were Messrs. Horace Lissenden and Partners, and the consulting engineers Messrs. J. C. Hughes & Partners. Messrs. Walter Lilly & Co., Ltd., were the main contractors, and the floor and roof were erected by the Vibrated Concrete Construction Co., Ltd., in a little more than three months.

Precast Lattice Girders.

A BUILDING recently constructed over the river Gala at Galashiels is carried on seven precast reinforced concrete lattice girders (*Fig.* 1), each comprising three sections joined together by concreting the projecting reinforcement (*Figs.* 2 and 3). The span of the river is more than 60 ft. and the girders are spaced at 6 ft. 3 in. centres. The structure is faced with brick, and the floor and roof are insulated. The floor comprises reinforced precast slabs 3 in. thick at the bottom, followed by 1 : 3 cement-sand mortar ½ in. thick on which are bedded insulating slabs 2 in. thick and there is a topping of granolithic 1½ in. thick. The roof comprises precast reinforced slabs 3 in. thick at the bottom, followed by insulating slabs 2 in. thick, a layer of 1 : 4 cement-sand mortar 1½ in. thick, and a roofing-gravel finish. The floor and roof slabs are supported on the

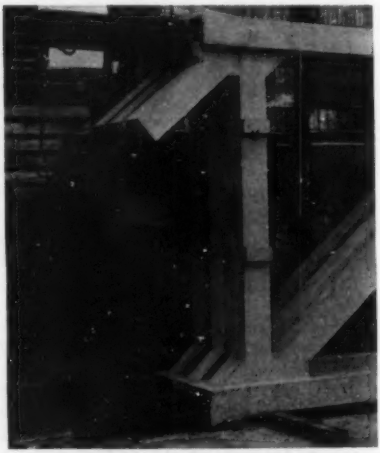


Fig. 2.—Projecting Reinforcement at Joint.

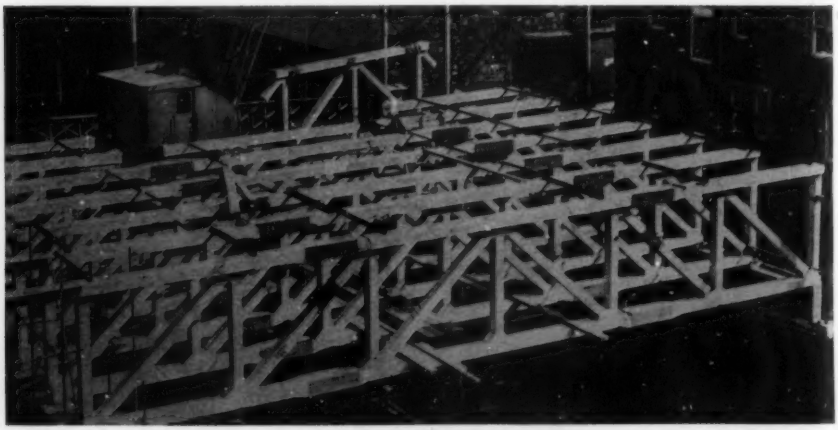


Fig. 1.—Erection of Girders.

bottom and top rails of the lattice girders, which are 9 ft. high.

The building is to be used as sweating houses for steaming wool off sheepskins, and reinforced concrete was used because of the risk of corrosion. The girders were precast because the possibility of flooding required that the falsework be in the river bed for as short a time as possible. Rails for hanging the skins are carried by intermediate lattice girders.

The consulting engineers were Messrs. Kinnear & Gordon, and the girders were made by the Scottish Construction Co., Ltd., at their factory in Edinburgh and transported thirty-five miles to the site.

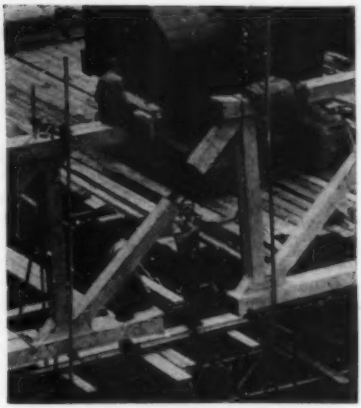


Fig. 3.—Joining the Girders.

"Concrete and Constructional Engineering" Prize Design.

THE prize of £25 awarded each year by Concrete Publications Limited for competition amongst the post-graduate students attending the course of concrete technology at the Department of Civil Engineering, Imperial College of Science and Technology (University of London), has been awarded for the session 1955–1956 to Mr. G. L. Evans. The competition, which was set by Professor A. L. L. Baker, D.Sc., was for a scheme for the reconstruction of an area of the docks on both sides of the river near Tower Bridge. It was to be assumed that all existing construction more than twenty-five years old or badly designed could be demolished and the area complete lyre-planned. The density of new buildings had to be equivalent to four stories covering all the land. The spacing and height of the buildings was to be arranged to give the best grouping from the points of view of appearance, lighting, and communication.

The structures included were: (1) A railway station on the south bank of the Thames connected by a viaduct to the existing railway system near London Bridge station. (2) A bridge over the Thames carrying four lines of traffic. (3) Jetties to accommodate the largest type of vessel at present berthed between Tower Bridge and London Bridge. (4) Flats, offices, warehouses up to 25 stories high with ample space between them, schools, public halls, and a church. Designs could be included for water tanks on high buildings and raft foundations for poor soils.

The assessor was Mr. Leslie Turner, B.Sc., M.I.C.E., who awarded the prize to Mr. Evans for his design for a railway station. The assessor's report states that the general standard of work, and the students' conception of the use of reinforced concrete, were very high, and that nearly half of the other designs submitted in the competition would have tied for second place.

Warehouse at Kingston-on-Thames.

A WAREHOUSE for Messrs. Bentalls, Ltd., at Kingston-on-Thames, completed last year, comprises four floors and a receiving and delivery yard. The structure is bounded on all sides by existing buildings, generally two stories high, built of load-bearing brickwork from which the new building had to be entirely separate.

All the floors were designed for an imposed load of 150 lb. per square foot, with the exception of the mezzanine floor over the loading dock which was designed for an imposed load of 100 lb. per square foot. The slabs are 5 in. thick and are carried on main beams supported on columns on a grid 20 ft. 6 in. by 26 ft. 6 in.

The secondary beams are at the third-points of the spans of the main beams.

To enable the business of the store to be carried on and vehicles to use the existing loading dock, three frames with a minimum headroom of 14 ft. were used in the yard. At first-floor level these frames carry the loads from the upper columns at about their third-points; the maximum calculated load from each column was 220 tons. The frames have spans varying from 54 ft. to 46 ft. Hinges are formed at the feet by reducing the width of the legs to 9 in. and providing inclined and crossing reinforcement bars. Temporary reinforcement bars at the hinges were provided near the outer faces to ensure that they were stable until the beams were built. The bars were then cut by oxy-acetylene flame and the filling of the space on either side of the "neck" delayed as long as possible so as to permit freedom of deformation of the frames in their hinged condition when most of the dead load was present. The beams are 5 ft. wide and 4 ft. 11 in. deep. At the end nearer the loading dock each beam is cantilevered beyond the leg to support a column from the upper floors.

The secondary beams for the first floor were precast with projecting reinforcement to bond with the floor slab; these beams were hoisted into position and bear on recesses left in the frames and the main beams. Adjustable centering to the floor slabs was used so as to avoid props in the yard.

Bore-holes revealed 8 ft. of made-up ground over 9 ft. 6 in. to 13 ft. 6 in. of gravel and 6 ft. of brown clay overlying London blue clay. The foundations are on the gravel, and a bearing pressure of 2·7 tons per square foot was adopted in the design. As the ground-water level was about 8 ft. below yard level, plain concrete was placed in the lower portion of the excavations, and the reinforced concrete work commenced at about 6 ft. below yard level. Where possible separate bases were used and these were combined with strip foundations where the external columns were alongside existing brick walls so that the foundations did not have to pass under the brick walls.

To provide access from the warehouse to the store, a reinforced concrete bridge, 46 ft. long, comprising three slabs spanning 8 ft. between the walls which formed the main longitudinal girders, was built. These walls, with large areas of window, are 5 in. thick, increasing to 9 in. between the windows to form vertical ribs.

The architects were Messrs. T. P. Bennett & Son, the consulting engineers were Considère Constructions, Ltd., and the main contractors Sir Robert McAlpine & Sons, Ltd.

Prestressed Sludge-Digestion Tanks.

THE sludge-digestion tanks (*Fig.* 1) for the Borough of Stafford sewage works are 60 ft. in diameter and have a capacity of 600,000 gallons each. They are believed to be the first prestressed sludge-digestion tanks in this country. The Magnel-Blaton system was used throughout. In one tank the sludge is heated and the gas used for the generation of power for the works. The other is constructed so that plant for making use of the gas can be added in future.

The conical floors are reinforced concrete 12 in. thick and were cast without shuttering at a slope of 30 deg. At the top of the cone is a ring-beam on which the prestressed wall was built. The sliding joint between the beam and the wall was painted with bitumen enamel, and incorporates a rubber waterstop 9 in. wide in the form of a complete ring, the joint being vulcanised before delivery.

The wall is 30 ft. high and 8 in. thick, and was cast in 4-ft. lifts in flexible steel shuttering. A continuous corbel was formed 4 ft. 9 in. from the top to support the steel gas-bell. Mild steel reinforcement was used only in the lower 8 ft. of the wall and at the top for reinforcing the corbel and cantilevered walkway.

The wall was prestressed vertically by cables, each containing eight 0·2-in. wires, at 3 ft. centres, anchored by bonding with the concrete at the bottom of the wall (*Fig.* 2). The cables were contained in steel sheaths with tubes of $\frac{1}{4}$ in. diameter welded to the foot for grouting purposes.

The circumferential cables were suspended on the face of the wall by hangers of 16-gauge mild steel strip. Cables comprising four wires of 0·276 in. diameter were used on the lower part of the wall and cables comprising four 0·2-in. wires towards the top. The 0·276-in. wires were tensioned at four points and the 0·2-in. wires at two points. The jacks (*Fig.* 3) were suspended from the top of the wall on a block-and-tackle and co-ordination of the tensioning was achieved by the use of telephones.

Two openings of 2 ft. 3 in. by 1 ft. 3 in. occurred near the bottom of the wall where the cables were spaced at 4-in. centres. The cables were diverted above

Fig. 1.

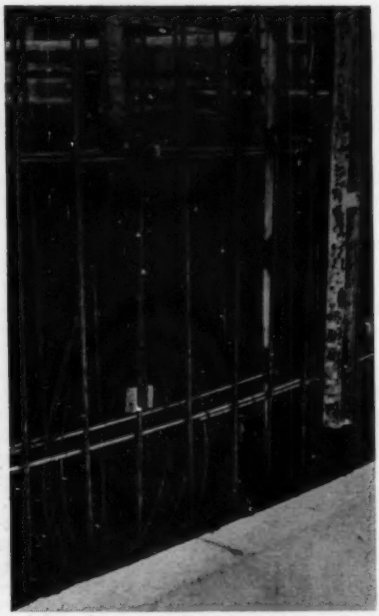


Fig. 2.—Bottom Anchorage of Vertical Cables.

Fig. 3.—Jacks for Tensioning Horizontal Cables.

and below the opening, each cable passing over or under a bolt cast in the wall (*Fig.* 4).

After stressing, the tanks were filled with water and the outside coated with gunite to provide a cover of 1 in. to the wires and anchor-blocks. The cantilevered walkway 3 ft. wide and the heating and other external chambers are in reinforced concrete. The reinforced concrete walls of the chambers, where they are in contact with the prestressed wall, are connected by mild steel bolts of $\frac{5}{8}$-in. diameter built in during the concreting of the main tank.

Mr. T. H. Higson, B.Sc., A.M.I.C.E., is the Borough Engineer. The contractors were Messrs. Wilson Lovatt & Sons, Ltd. The guniting was done by the Concrete Proofing Co., Ltd., and the design of the tanks was by the British Reinforced Concrete Engineering Co., Ltd.

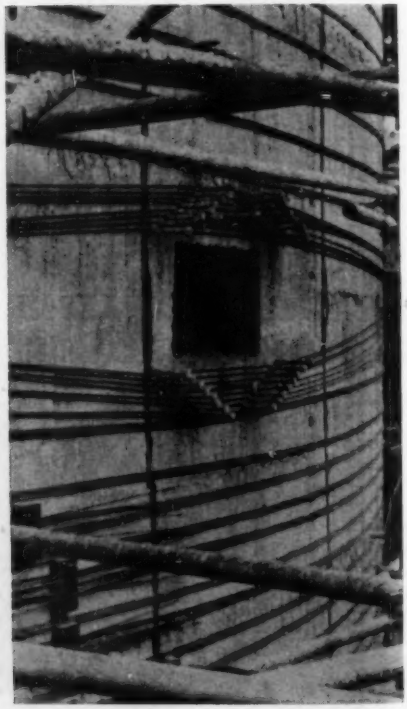
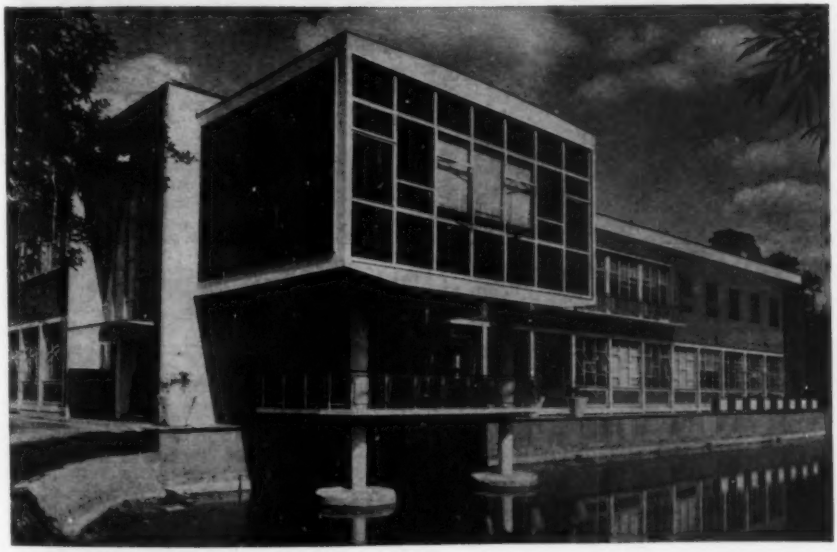


Fig. 4.—Arrangement of Cables at Openings.

Laboratories at Egham, Surrey.

THE laboratories (illustrated below) at Egham, for Shell Chemicals, Ltd., completed in 1955, are contained in a two-story U-shaped building about 160 ft. by 120 ft. overall in plan. The plan was determined by the standard laboratory bench with its services, and these also dictated the constructional forms used. The construction comprises reinforced concrete columns, peripheral beams, suspended beam-and-slab ground floor, hollow-tile first floor, and aluminium-trough roofing. The first floor and roof are supported on light welded-steel trusses about 2 ft. 6 in. deep in order to provide a deep ceiling space with little obstruction to the passage of service pipes of large diameter. The end members of the trusses are bolted to the sides of the columns, into which had been built steel side-plates connected by tubes through which the bolts were inserted.

The main entrance unit and conference room are reinforced concrete box-frames



which together provide a degree of stiffness to the structure. The conference room projects over the lake and is also supported by two circular reinforced columns. The interior contains a main staircase with central reinforced concrete stringers supporting teak treads bolted to the stringers.

The architects are Messrs. Walker, Harwood & Cranswick, F./A.R.I.B.A., the consulting engineer is Mr. Maurice Nachshen, B.Sc., M.I.C.E., M.I.Struct.E., the quantity surveyors Messrs. Harris & Porter, and the contractors Messrs. Trollope & Colls, Ltd.

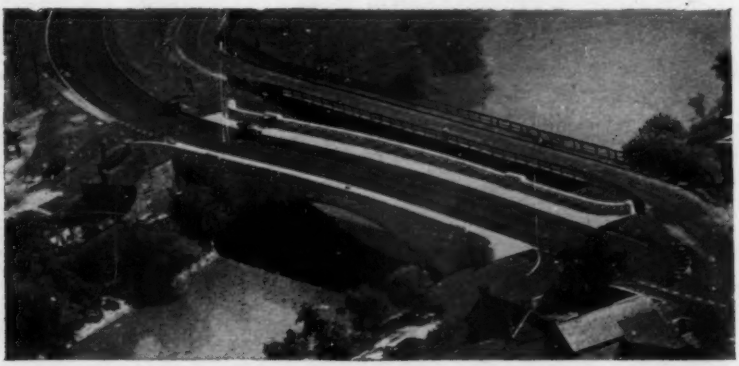
Bridge over the River Severn.

MAISEMORE bridge over the river Severn, near Gloucester, was opened to traffic in June, 1956. This work was started in 1939 and re-started as a second contract late in 1953.

The abutments are of plain concrete in the proportions of 1 : 3 : 6. The bridge is a single-span reinforced concrete arch with stone facing to the spandrels, abutments, and wing-walls, and with stone parapets throughout. The clear span (measured on the skew) is 150 ft. and the width between the parapets 45 ft. The rise of the arch is 18 ft. Permanent hinges at the abutments were constructed, using 1½-in. steel bars and helical reinforcement to distribute the high stresses.

The bridge consists of seven arch ribs 9 in. deep with transverse stiffening ribs parallel with the banks of the river. The vault slab varies from 8 in. thick at the springings to 4 in. at the crown. The deck slab is 8 in. thick.

The engineer for the Gloucestershire County Council is Mr. E. C. Boyce, County Surveyor, the architects were Messrs. H. Stratton Davis & Yates, the consulting engineers were Considère Constructions, Ltd., and the contractors Messrs. A. E. Farr, Ltd.



Maisemore Bridge, over the River Severn.

Prestressed Concrete Development Group.

THE annual general meeting of the Prestressed Concrete Development Group is to be held on January 24 at the Institution of Civil Engineers, Great George Street, London, S.W.1. Following the meeting a paper entitled "Progress of Prestressed Concrete in the Netherlands" is to be presented by Ir. A. S. G. Bruggeling, who is chief engineer of Spanbetonmij, prestressed precast concrete manufacturers in the Netherlands. Tickets for non-members of the Group are available (free of charge) from the Prestressed Concrete Development Group, 52 Grosvenor Gardens, London, S.W.1.

Two Bridges in the Gold Coast.

THE bridge over the river Volta (*Fig.* 1) comprises a steel arch carrying a road 22 ft. wide over the river and three reinforced concrete approach spans, each 50 ft. long, on each side of the river. Because the piers are founded on rock and will have very little settlement, the longitudinal beams, which are 4 ft. deep at the middle of the span, are continuous over the two intermediate supports. High-tensile steel reinforcement was used. The compressive strength of the concrete was specified to be 3750 lb. per square inch at 28 days. Some details are shown in *Fig.* 3. The consulting engineers are Sir William Halcrow & Partners and Messrs. Freeman, Fox & Partners. Messrs. Dorman Long (Bridge & Engineering), Ltd., were the contractors.

In *Fig.* 2 is shown the reconstruction of a bridge with a timber deck at Kpeshi lagoon. The work comprised the construction of reinforced concrete transverse beams, with cantilevered ends, on the existing piers and abutments and the erection of prestressed precast beams of 30 ft. span between the transverse beams. The carriageway is 22 ft. wide and there are two footpaths each 5 ft. wide. The deck was designed for full Ministry of Transport loading. In spite of the cost of transport, it was cheaper to have the

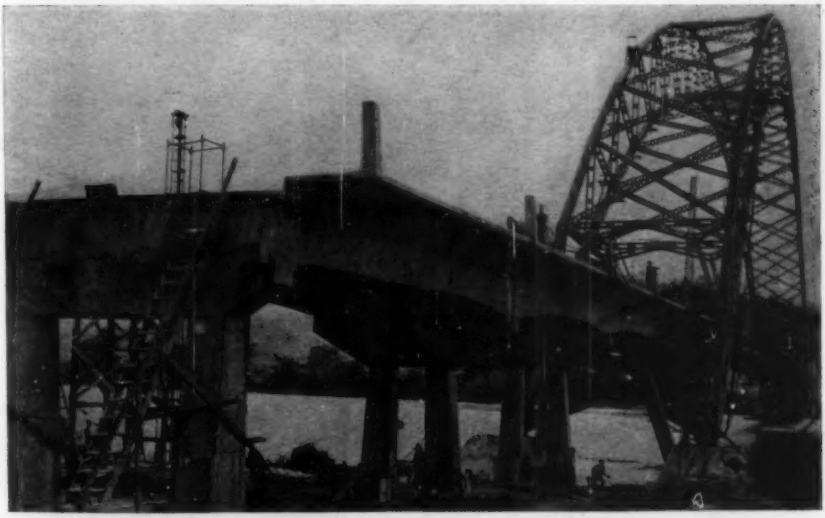
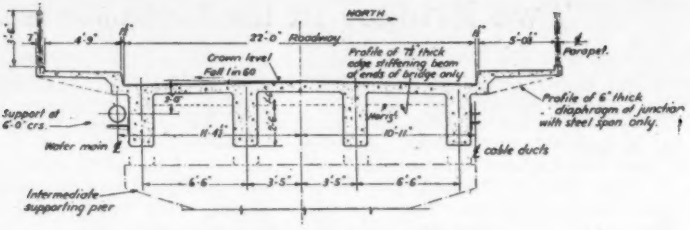


Fig. 1.—Approach to Volta Bridge, Gold Coast.

Fig. 2.—Bridge on Kpeshi Lagoon, Gold Coast.



TYPICAL CROSS SECTION OF BRIDGE.

Scale. ¼ in. to 1ft.

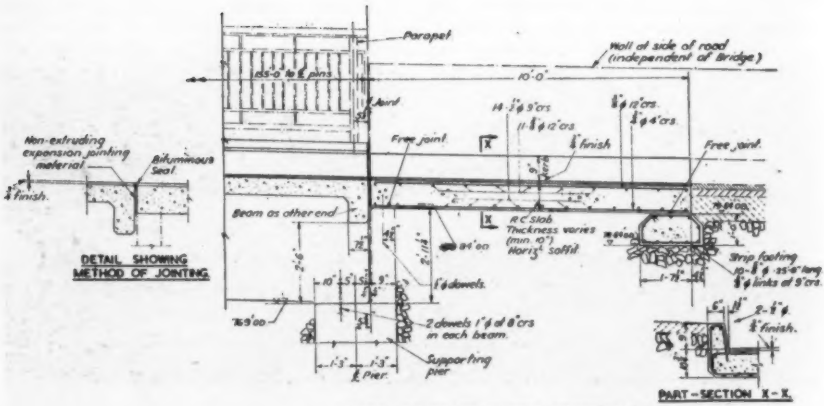


Fig. 3.—Details of Bridge over the River Volta.

prestressed beams made in England than in the Gold Coast. The lower flanges of the beams are contiguous, and the top flanges are bonded to the in-situ concrete filling between the webs. The consulting engineers are Sir William Halcrow & Partners. The contractors were the New Gold Coast Building Co., Ltd., and the prestressed beams were made by Anglian Building Products, Ltd.

Both bridges were built for the Director of Public Works of the Gold Coast.

The Production of Rocket Missiles.

THE British Oxygen Co., Ltd., and Messrs. George Wimpey, Ltd., have formed British Oxygen Wimpey, Ltd., for the construction of rocket-missile projects. The experience of these firms in building construction and in low-temperature methods of producing liquid gas will enable all phases of such projects to be dealt with by the new company, whose office is Bridgewater House, Cleveland Row, London, S.W.1. The British Oxygen Co., Ltd., and Messrs. George Wimpey, Ltd., had been awarded a contract by the Ministry of Works for the erection of buildings and plant for dealing with liquid oxygen and nitrogen, together with laboratories, workshops, and administration buildings, at the rocket-missile establishment at Spadeadam Waste, Cumberland.

Factory at Dronfield, Yorkshire.

A FACTORY (*Figs.* 1 and 2) for the manufacture of tools has been recently completed at Dronfield for Messrs. Padley & Venables, Ltd. About three-fifths of the site is made-up ground, and piling and compacted filling are used as foundations over this area. Short bored piles were used owing to the comparatively shallow depth of the load-bearing rock ; 286 piles were driven varying in depth from 10 ft. to 17 ft. and in diameters of 1 ft. 4 in. to 2 ft. 6 in.

Over the filled area the frame is wholly supported on piles. The floor slab is isolated from the pile caps and supported on a compacted filling. The factory is divided into three main parts, namely a bar-storage building 200 ft. long by 30 ft. wide, a main building of 82,000 sq. ft. in four bays of 50 ft., and a two-story administration building. *Fig.* 3 shows one of the bays erected, and *Fig.* 4 the casting on the site of the components of the frames.

The bar-storage building comprises reinforced concrete precast frames of 30 ft. span at 16 ft. 8 in. centres with a height to eaves of 28 ft. 6 in. over 100 ft.

Fig. 1.—General View.

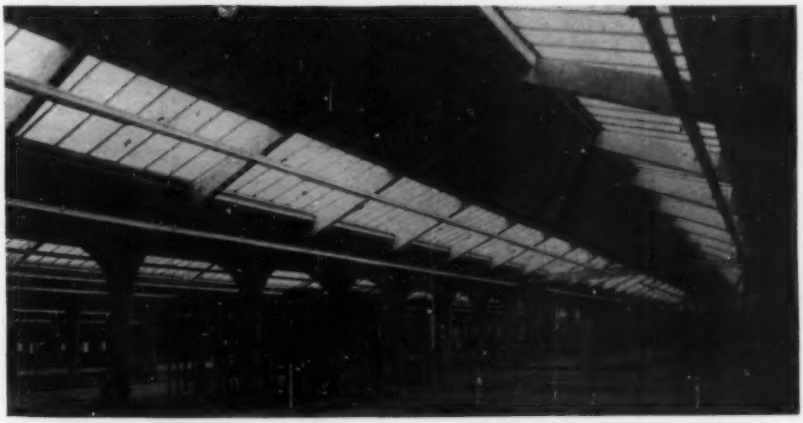


Fig. 2.—The Main Shop.

of its length and 15 ft. in the other 100 ft. Over the full length of the higher end provision is made for a 3-ton overhead electric crane supported on reinforced concrete precast beams.

The main factory has reinforced concrete precast frames spanning 50 ft. at 15 ft. centres supporting precast concrete purlins and asbestos-cement sheets. Precast valley and eaves beams provide longitudinal stability to the structure, and a complete expansion joint is incorporated through the centre. The frames are designed to allow for an additional imposed load of 1 ton suspended from any point.

Foundations for Hammers.

A special requirement of the clients was the isolation of the forging hammers

Fig. 3.—A Bay During Construction.

Fig. 4.—Precasting the Frames.

from the structure and the floor. This was achieved by designing the foundation on a two-mass principle, with a layer of high-density cork; the results proved to be satisfactory.

The foundations for the hammers are of the type shown in *Fig.* 5 and, to prevent vibration, consist of two masses. The upper mass comprises the hammer, anvil, and inner concrete block. The lower mass is the outer concrete that rests on the soil. Between the concrete blocks is a pad of cork of special grade 2 in. thick. The soil beneath the lower block was also considered as being in the nature of an elastic support.

The two-mass foundation was used to provide the necessary degree of isolation from shock and vibration causing both vertical and lateral displacements. Cork was used because it can be of any thickness and permissible pressure, and because it is easily handled. Cork also has the advantage that it does not reflect shock waves at the junction of the inner block, that is there is no transmission of shock from cork to concrete. This is perhaps the most important reason for using the two-mass arrangement, since if a single mass resting on the soil were used the transmission of shock from the soil would be from 40 per cent. to 100 per cent. depending on the nature of the soil; say 50 per cent. for average soils.

For the purpose of assessing the force transmitted to the outer block it was assumed that the block does not move during the very short period of the blow, which was calculated to be 1/576 second. Consequently the motion of the inner block only is considered, and its initial velocity was calculated from the simple momentum relationship, from which the maximum displacement of the inner block, and hence the force transmitted, can be found. In the case of the 5-cwt. hammer the transmitted force was 50,000 lb., which is ninety times the weight of the tup. As a check the transmission of the force was assessed by impulse analysis.

All the hammers had a blow frequency of 2 to 4 cycles per second, and it was not practical to design a foundation with a natural frequency substantially less than the frequency of the blow. It was therefore decided to arrange for the natural frequency of the foundation to be well above the frequency of the blow so that resonance would be impossible, and to ensure that sufficient cycles of vibration occurred between blows to permit the vibration to be damped out. In no case was the number of cycles of vibration between blows less than five. For the cork used the damping for five cycles is 86·5 per cent. This is a theoretical degree of damping, and is likely to be less than that realised in practice, and no account has been taken of the frictional damping of the cork on the vertical sides of the inner block.

An important aspect of the foundation is the transmissibility in terms of the amplitude of the vibration. Although it is important to know what

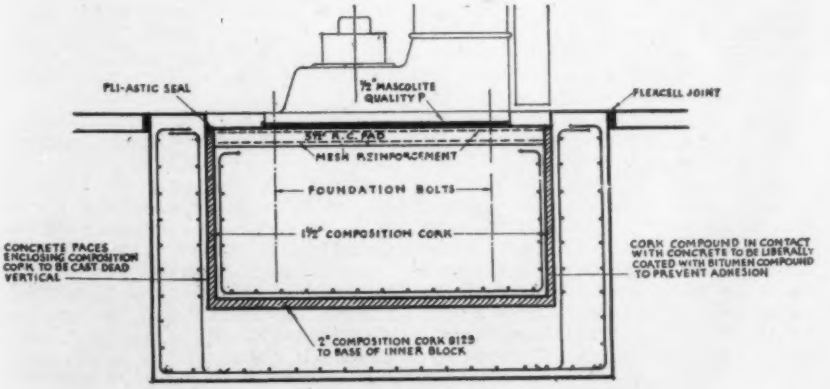

PLI-ASTIC SEAL
½" MASCOLITE QUALITY P
FLEXCELL JOINT
MESH REINFORCEMENT
FOUNDATION BOLTS
1½" COMPOSITION CORK
CONCRETE FACES ENCLOSING COMPOSITION CORK TO BE CAST DEAD VERTICAL
CORK COMPOUND IN CONTACT WITH CONCRETE TO BE LIBERALLY COATED WITH BITUMEN COMPOUND TO PREVENT ADHESION
2" COMPOSITION CORK BED TO BASE OF INNER BLOCK

Fig. 5.—Foundation for Forging Hammer.

force is transmitted, the criterion is really the amplitude of vibration of the foundation on the soil. The amplitude transmissibility was calculated from expressions derived for a two-mass system. The results obtained would be high since they were for steady vibrations. Since the vibration was initiated by an impulsive force there would be a further reduction in amplitude of the outer block in contact with the soil due to momentum.

The machines are mounted on sheets of mascolite ½ in. thick in order to prevent local overstressing of the inner concrete block due to surface irregularities, and to reduce the intensity of the stress-wave transmitted to the foundation by the anvil. The pads of reinforced concrete 5½ in. thick immediately below the machines were cast after the main body of the blocks had hardened; this facilitated truing the bearing surfaces and matching them to the bases of the machines.

All the external walls consist of precast concrete panels made by the Reema process. The panels are 7 ft. 6 in. wide and generally of story height, comprising two leaves 1½ in. thick with vertical webs spanning a 6½-in. cavity and with exposed-aggregate external finish. Door and window frames were incorporated in the panels, which were made in a factory and so reduced construction time on the site. Crushed gravel and multi-coloured quartzite were used for the exposed-aggregate finishes for the main factory building, and crushed white Derbyshire spar for the administration building.

The administration building is of reinforced concrete frame construction cast in situ, with a roof of Stahlton prestressed concrete slabs. This building also accommodates on the ground floor a kitchen and works canteen and on the first floor a staff canteen.

The electrical power distribution within the factory is by means of eight bus-bars extending the full length of each bay, and lighting is by continuous trunking using fluorescent tubes. Mains for water, gas, and compressed air are supported by the frames. Clearing and levelling of the site was commenced in September, 1954, and the work was completed in July, 1956.

The engineers and architects were Messrs. C. W. Glover & Partners. Messrs. Henry Boot & Sons, Ltd., were the main contractors, with the following sub-contractors: Precast frames, Messrs. William Prestwich & Son, Ltd.; pile boring, Normid, Ltd.; precast walls, Messrs. Henry Boot & Son (Reema), Ltd.; prestressed roof slabs, the Costain Concrete Co. Ltd.

Management in the Building Industry.

A "STUDY GROUP" of the British Institute of Management, comprising representatives of the building and civil engineering industries under the chairmanship of Sir Hugh Beaver, K.B.E., which was appointed last year to study the question of training men for managerial positions in these industries, has now issued its report ("Management Training in the Building Industry". Price 5*s*.). Fifteen pages of generalisations on the need for training in management are followed by a recommendation that it would be desirable to have a "staff college" for the industry, and that in the meantime there should be made available a course in management of not more than one week's duration for men about twenty-eight years of age who have completed their technical training, and a course of about four weeks' duration for men of thirty-five and over who appear to be suitable for promotion to positions of responsibility. The study group seems to have little faith in young men. This apparent reluctance on the part of the present managers to entrust men with responsibility until they reach middle age is disappointing, particularly in an industry in which some of the largest businesses in existence to-day were founded by young men. Attendance at special courses is no doubt valuable. The report, however, seems to ignore the best method of all, which is for a manager himself to teach promising young men in his own business, and above all to thrust responsibility upon them at an early age. Any industry will become stagnant if no one is considered for a managerial position until he is at least thirty-five years of age.

Office Building, Lagos, Nigeria.

THE new office building for the Shell Company of West Africa was completed in 1955. It consists of two main structures, namely a five-story office building (*Fig.* 1) measuring about 115 ft. by 42 ft. in plan and a smaller service building about 75 ft. by 27 ft. in plan which contains only lifts, staircases, and toilets. This service building is about 15 ft. to the rear of the office building and is linked to it at all floor levels by corridors in the form of reinforced concrete "bridges" supported on bronze rocker and sliding bearings to allow for differential settlement between the two buildings. It is intended later to construct a second office block behind the service building and connected to it by similar bridges.

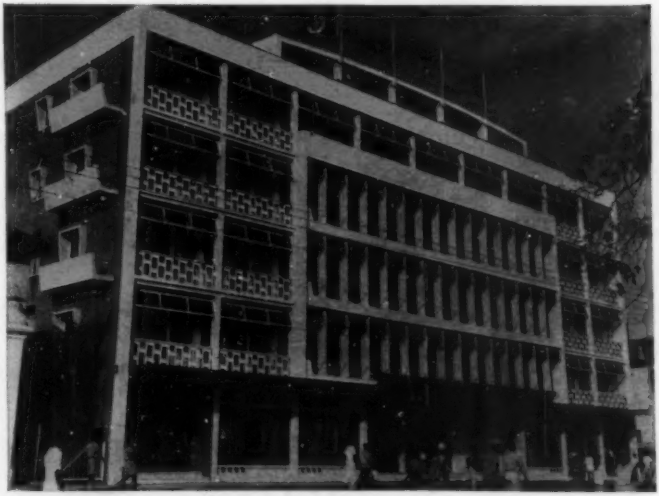


Fig. 1.—Main Elevation.

The superstructure of the office building is of reinforced concrete with solid floors. A high degree of flexibility was required in the positions of partitions, so that ordinary beam-and-column construction was undesirable. The beams are therefore wide and only 1 ft. deep and span in the longitudinal direction only at the front and rear elevations and along the centre of the building. The building has thus a uniform cross section throughout its length, permitting subdivision by standard partitions at any window mullion. The columns are in four rows, that is one row at the rear, one row at the front, and two rows supporting the central beam. The floor slabs, which span transversely between the beams, are designed to resist wind forces as well as vertical loads. Additional transverse stiffness is provided by the end walls, which are of reinforced concrete with only a few windows. The front and rear walls above ground floor are almost completely window space, and sun-glare is reduced by balconies, vertical fins, and screens of tinted glass.

The service building is of ordinary framed construction.

Both the office and the service building are supported on cellular raft foundations. The subsoil is swamp to depths beyond the economical reach of piling, and consists of fine sand and silt with soft clay lower down. Ground-water level is about 2 ft. 6 in. below the surface. A semi-buoyant cellular reinforced concrete raft was designed so as to impose a load of not more than $\frac{1}{2}$ ton per square foot in excess of the weight of soil excavated. It

was essential that the raft be of considerable stiffness to resist the bending moments, but it was also important that its weight should be small. The raft is 7 ft. deep overall with top and bottom slabs 8 in. and 10 in. thick and 9-in. and 10-in. ribs.

Local aggregates and British cement were used. The concrete was made under careful control and good test-cube and test-beam results were obtained.

The architects are Messrs. Walker, Harwood & Cranwick, F./A.R.I.B.A., the consulting engineer Mr. Maurice Nachshen, B.Sc., M.I.C.E., M.I.Struct.E., the quantity surveyors Messrs. Harris & Porter, and the contractors Borini-Prono (Nigeria), Ltd.

Residential Flats, Eaton Terrace, Westminster.

UNUSUAL DESIGN OF FLOORS.

FLOORS of unusual design are a feature of a nine-story block of flats approaching completion at the corner of Eaton Terrace and Ebury Street, London, S.W.1, which is one of several being built for the Metropolitan Police. The structure (*Fig.* 1) will contain fifty-two flats for police officers and their families, as well as a public house with its own flat and two shops to replace those previously on the site.

The main building is T-shaped; the head of the T is 158 ft. long and 30 ft. wide with access balconies at the rear, and the leg is 50 ft. by 31 ft. by seven stories high. The leg contains stairs and lifts serving the whole building, and there is one flat on each flocr. Three flats with drying-rooms and other amenities in an extended basement are in a separate building facing the side street.

The buildings are founded on sandy ballast about 10 ft. below street level. The superstructure is of reinforced concrete columns, peripheral beams, and cross beams at the party walls between the flats, but there are no beams within any flat. There are two internal columns at the corners of rooms in each flat. The floors are 8 in. thick (5 in. of reinforced concrete and 3 in. floor finish), except at the columns where upstands 2 in. high are added to the structural slab for an area 5 ft. 6 in. square (*Fig.* 3). These perform the function of dropped panels in flat-slab construction. The floor finish

Fig. 1.—Main Elevation during Construction.

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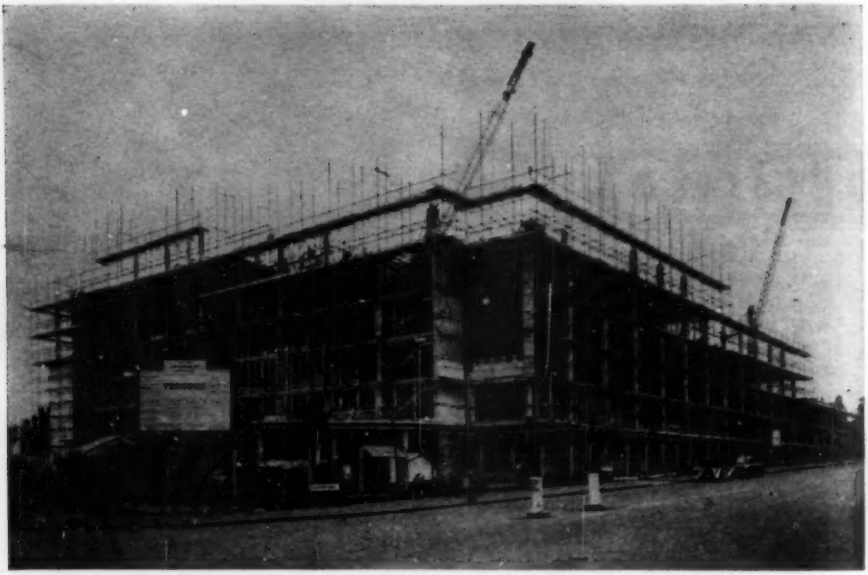

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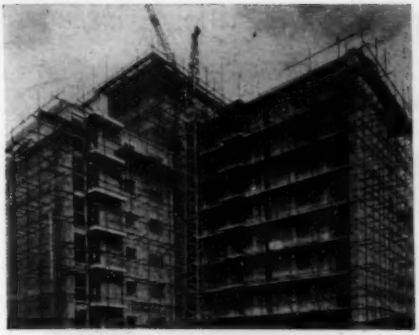

Fig. 2.—Side Elevation.

Fig. 3.—Thickened Floor at Columns.

over the upstands is reduced to 1 in. By limiting the floor thickness to 8 in. and eliminating both dropped panels and beams, the floor to floor height was kept down to the required 8 ft. 3 in.; the clear height of 7 ft. 6½ in. is found to be adequate.

The architect is Mr. J. Innes Elliot, B.Arch., A.R.I.B.A., Chief Architect and Surveyor of the Metropolitan Police, and the senior architect in charge was Mr. D. T. Edwards, A.R.I.B.A. The consulting engineer is Mr. Maurice Nachshen, B.Sc., M.I.C.E., M.I.Struct.E., the quantity surveyors Messrs. F. J. Meekins & Partners, and the contractors Messrs. Robert Hart & Sons, Ltd.

International Congress on Prestressing.

THE next congress of the Federation Internationale de la Precontrainte will be held in Berlin from 5 to 10 May, 1958, when the following subjects will be discussed.

1. Development in design methods. General reporter: Prof. Dr. Ing. H. Rüsch, Munich.

2. Advances in site work, with particular reference to grouting, anchorages, reduction of friction, and safety measures. General reporter: Dr. Ing. G. F. Janssonius, Amsterdam.

3. Progress in the manufacture of prestressed concrete in factories and the use and assembling of precast units on the site. General reporter: Mr. D. H. New, London.

4. Structures built wholly or partly in prestressed concrete in which important developments in design or construction have been included. General reporter: M. P. Lebelle, Paris.

Further information may be obtained from Mr. P. Gooding, Cement and Concrete Association, 52 Grosvenor Gardens, London, S.W.1, not later than 1 August, 1957.

Grouted Concrete.

ONE of the advantages of grouted concrete is the ease with which it can be placed under water, so that in marine construction delays due to tides can be avoided. An example is the strengthening of a wharf at Rochester to carry cranes weighing 10 tons. The work included the construction of thirty-nine piers 5 ft. 11 in. by 1 ft. 8 in. in cross section by 16 ft. 8 in. high, and thirty-eight beams 3 ft. 6 in. by 1 ft. 9 in. in cross section by 8 ft. 4 in. long, all in reinforced concrete. A total of 290 cu. yd. of concrete was used, and the cost of supplying and placing the aggregate and grouting it with mortar was £1542. When the reinforcement was fixed, the coarse aggregate graded from 3 in. to ¾ in. was placed in the shutters at any state of the tide, and the mortar applied through a pipe with a bore of 1 in. as shown in *Fig.* 1. *Fig.* 2 shows the plant used for the preparation of the mortar. The engineers were Sir William Halcrow & Partners, and the general contractors the Demolition & Construction Co., Ltd. The sub-contractors for the concreting of the piers and beams were Messrs. Edmund Nuttall, Sons & Co. (London), Ltd., who are licensees for the "Intrusion-Prepakt" process used. Some notes on this process follow.

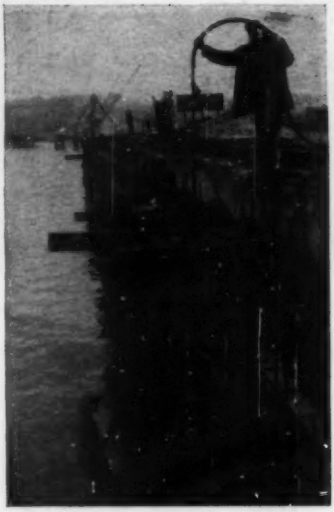

Fig. 1.—Placing the Grout.

Fig. 2.—The Mixing Plant.

The coarse aggregate is compacted by external vibration of the shutter or by internal vibration, or in some cases, such as the piers at Rochester, scaffold tubes or similar tampers may be used. Compaction of the aggregate reduces the voids in the coarse aggregate and so effects a saving in the quantity of mortar used. It also increases the crushing strength of the concrete by up to 10 per cent. The economy of consolidating the aggregate depends on the cost of carrying it out compared with the cost of the mortar saved. In large sections the aggregate settles fairly well as it is placed and the reduction of voids due to compaction is seldom worth the cost of the labour required. In smaller sections about 10 per cent. additional coarse aggregate can be placed in the shutters if the aggregate is rodded or vibrated as it is placed. Poor consolidation of the coarse aggregate results in only a slightly lower crushing strength, but the quality of the resulting concrete remains good. For sections of 4 in. or less in thickness the following gradings are commonly used: Passing a ¾-in. sieve, 100 per cent.; passing a ½-in. sieve, 85 per cent.; passing a ⅜-in. sieve, up to 10 per cent. The spacing of

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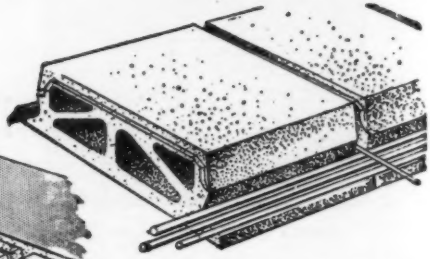

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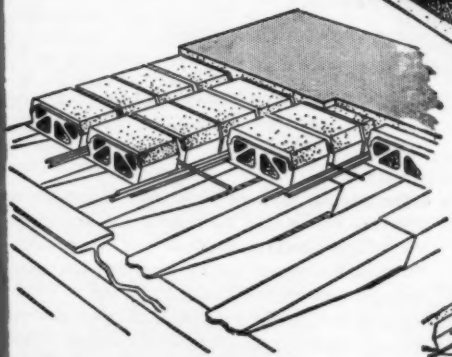
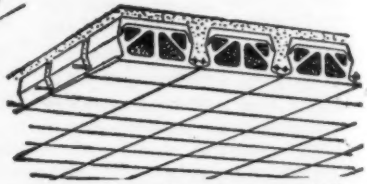


Showing Two-Way Reinforcement and Hollow Concrete Blocks laid on Trianco Telescopic Centers.



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reinforcement bars and the cover of concrete are the same as in the case of ordinary concrete work.

The quantity of mortar used per cubic yard depends on the voids in the coarse aggregate. If large aggregate is used, say 9-in., the voids will be about 35 per cent., whereas with smaller aggregate the voids may be as high as 45 per cent. An average volume of voids is 42 per cent. In 1 cu. yd. of concrete there will be 1 cu. yd. of coarse aggregate and a volume of mortar represented by the voids in the aggregate.

The mortar is introduced at the lowest point in the shuttering, either through a hole in the side or bottom of the shutter or through a pipe pushed vertically down through the coarse aggregate. In the case of San Esteban dam in Spain these vertical pipes were pushed down through 60 ft. of coarse aggregate before pumping of the mortar started. As the pumping proceeds a large " bubble " of mortar is formed at the bottom which continues to swell upwards and outwards and completely fills the voids. If vertical grouting pipes are used they are raised as the mortar rises but never above the top of the mortar already placed. When horizontal pipes are used they are moved up the side of the shutter, keeping them below the top of the mortar already present. The purpose of pushing the mortar upward through the aggregate is to ensure that every void is filled ; this can be checked by probes pushed downwards into the mass. Slotted pipes of 2-in. bore are also provided where necessary so that the level of the mortar can be noted. The consistency of the mortar is like that of enamel paint, although the water-cement ratio is between 0·40 and 0·47 for structural-grade concrete and from 0·45 to 0·58 for lean plain concrete.

The ingredients of the mortar are fine sand (fineness modulus 1·5 to 2), water, Portland cement, " Alfesil " (a very fine pozzolanic material which is cheap and in abundant supply), and chemicals that enable the water-cement ratio to be kept low, that increase flowability and penetrability, and that keep the solids in suspension whilst the mortar is hardening. These chemicals also prevent shrinking of the mortar during setting and control the setting time. Resistance to freezing and thawing is claimed to be equal to that of air-entrained concrete, and structural-grade concrete made by this process has a high degree of impermeability and resistance to sulphate action. These chemicals also ensure the retention of the mixing water during the hardening period and reduce the phenomenon known as " bleeding ". The ingredients are mixed in propeller-type tanks and then pumped into the shuttering ; the mortar has been pumped a distance of 2000 ft. in a single stage.

The normal range of strengths is between 2500 lb. and 4000 lb. per square inch at 28 days and from 3000 lb. to 7000 lb. per square inch at 90 days. The maximum range is from 500 lb. to 13,000 lb. per square inch at one year. Test cubes are made by first placing 1½-in. to ⅜-in. graded aggregate into 6-in. moulds and introducing the mortar through the bottom plate of the mould in the same manner as the work is being carried out on the site. The cubes can be taken from the moulds after twelve hours and cured under water.

Among the claims made for Prepakt concrete are the following. Cement content : normally, 2 cwt. to 4 cwt. per cubic yard ; minimum, less than 1 cwt. per cubic yard. Temperature rise : up to 60 per cent. less than ordinary concrete. Drying shrinkage : 50 per cent. less than ordinary concrete. Good bond to existing concrete. Resistance to action of salt water due to presence of a pozzolanic material in the mortar. If sand is omitted from the mortar a very penetrable grout results that can be used for consolidating formations and repairing badly-cracked structures. The rate of introducing the grout can be controlled to avoid the development of excessive hydrostatic pressures against the shutters.

Pumping can be stopped at any time and resumed several days later ; the joint between the two masses of mortar will be impermeable and impossible to detect by eye. No laitance is formed on the top surface.

Standard Forms for Proportioning Concrete Mixtures.

It is seldom that an engineer is responsible for the production of concrete in three continents and must deal with the differences of materials and conditions of operation that exist between Australia, England, and South America. To control manufacture under such different conditions it is necessary to have simple methods, to have them clearly described and explained, and to have standard methods of calculation and test set out on suitably-prepared forms so that equally good results should be attainable under a variety of working conditions. This is the task that has been undertaken by Mr. L. Boyd Mercer in recent years, resulting in methods described in his "Concrete Mix Design" published in Melbourne in 1953, summarised in "Ready-Mixed Concrete: Quality Control Requirements", read at the Symposium on Concrete Mix Design held in London in May, 1954, and again more fully treated recently in the American journal "Concrete" in an article entitled "Practical Method of Concrete Mix Design". The main difference between Mr. Mercer's method and a somewhat similar one introduced more than twenty years ago is described by the sub-title "A design based on provision for variables by standard forms".

The essential steps of the method are briefly as follows.

(1) The permissible minimum strength is the product of the working stress and the factor of safety. All, or almost all, the cube strengths will be above this, and their average will have to be above it by an amount to be decided; this average is called the "target strength". If the concrete is in connection with a prolonged project, such as a ready-mixed concrete plant, sufficient strength-test results may be available to enable a reliable coefficient of variation to be decided, and in such cases a statistical method may be used to determine the target strength. The application of the statistical method is made easy by a printed diagram of normal distribution curves, from which the relation of the "target" average strength to the minimum strength may be either read directly or easily determined by interpolation. Mr. Mercer emphasises that statistical control can have little direct application except on prolonged projects. He also indicates that he has used the coefficient of variation pending collection of data that may indicate the practicability of using a standard deviation.

(2) The next step is the determination by tests of the relationship between water-cement ratio and strength. Tests with at least three different water-cement ratios relating to a suitable range are made and the results plotted on a standard graph form. The water cement ratio corresponding to the required target strength is read from the resulting curve.

(3) The cement content corresponding to the required water-cement ratio, consistency, and maximum size and shape of aggregate is read from another diagram, which will be available for the aggregate in use at an established plant, or which can be prepared from the mixture used for establishing the relationship between the water-cement ratio and the strength.

(4) The weight of combined aggregate per cubic yard is now calculated by the absolute-volume method.

Mr. Mercer claims that the procedure in (3) and (4) is more convenient than using a diagram connecting the water-cement ratio and the proportion of aggregate to cement and reading from it the required cement content. This is true where batching by weight is used. For smaller works and batching by volume it is simpler to use a diagram that gives the number of cubic feet of aggregate per bag of cement, or the aggregate-cement ratio directly; less calculation is required by this method.

(5) The proportions of the different sizes of available aggregates are determined from the sieve analyses by selecting (again with the help of standard forms) the proportions that will give the best approximation to a type grading, or "basis" grading, suitable for making workable concrete with the calculated cement content.

(6) The proportions for a trial mixture are then readily calculated.

(7) A trial mixture is made and, if necessary, modifications in the proportions are introduced and further trials made to establish the final proportions.

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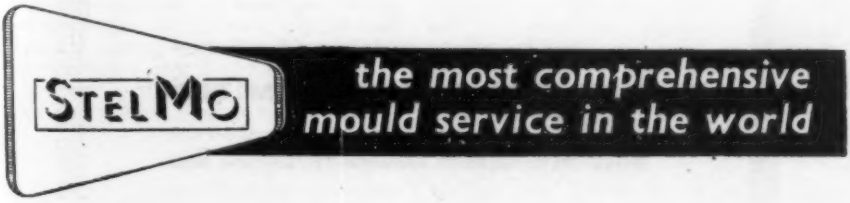

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Except in its use of statistics in the initial stage, this method is closely similar in principle and operation to one long in use in these countries, and in which the type gradings were not regarded as "ideal" gradings but as the coarsest continuous gradings that would make concrete that is just workable and as guides to establishing proportions of fine and coarse aggregates that would make somewhat more workable concrete without serious loss of quality. The new features are the simplified application of statistical methods to produce the "target" strength and the use of standard forms.

What is particularly interesting is the manner in which the whole procedure is made to appear so simple. In reference to the use of statistics Mr. Mercer points out that "practical" readers need not be deterred by it, and says: "It is no more necessary to understand the mathematical intricacies of statistical analysis in order to enjoy the benefits than it is to understand the operation of radio or television in order to be entertained. All the reader has to do is to learn to turn the correct knobs." Similarly Mr. Mercer has made the stages in the selection of proportions so simple that it would appear that one with little or no theoretical knowledge of concrete could operate the method with the help of the graphs and forms so as to produce concrete of the required quality. Possibly in dealing with aggregates having difficult gradings a man with no theoretical knowledge would find unexpected trouble in trying to get proportions of fine and coarse materials that would yield a combined aggregate having a grading close to the relevant type; and, if without experience or knowledge of the effects of departures from the type grading, he might have to make several trial batches before obtaining a satisfactory result. But it is perhaps wiser to gloss over these difficulties, which in any case will yield to persistence, and to make the method seem as easy and direct as it really is when the available aggregates are reasonably suitable for producing a combined aggregate with a grading near to the type.

Mr. Mercer has succeeded excellently in giving a clear picture of his method. His diagrams and tables should make standardised control successful in works in different countries; of course, changes of some units would be necessary between, say, England and the U.S.A., but this is a small matter. It should be appreciated too that, apart from the statistical approach, the method can be applied equally well on smaller and less permanent works than ready-mixed concrete plants. In works where batching by volume is used a set of curves showing relations of water-cement ratio to cement content to aggregate ratio (or to cubic feet of aggregate per bag of cement) would be more useful than his water-cement ratio to cement-content diagram. However, this is only a small difference of procedure.

Deciding the proportions for the mixture is only one stage of making uniform concrete. It must be followed by persistent control to ensure uniformity. This is emphasised by Mr. Mercer who mentions the factors that cause variations in compressive strength, their likelihood of occurrence, and the probable extent of their effects on strength. There are sixty items in this tabulation, of which three refer to cement, thirty-two to batching and mixing, and twenty-five to testing. This table would be useful to those engaged in inspection and control of concrete and particularly to suppliers of ready-mixed concrete, who must be able to assess the effects of changes in materials and the probable causes of variations in the strengths of test specimens.—H. N. WALSH.

A Dangerous Doctrine.

IN Building Bulletin No. 8, issued by the Ministry of Education and dealing with the use of precast concrete in the construction of schools, it is stated that three principles are essential in the use of prefabrication in multiple-story buildings. One of these principles is "that it [the system] should enable permanent buildings of good quality to be built (since permanence and quality depend on specifications and skill in design rather than on technique of production and erection, there is nothing incompatible between this aim and the requirement that prefabrication be used)."

The Ministry might usefully ponder the

question whether even the best design, depicted in the neatest of drawings and described in the most detailed specification, is likely to result in a permanent structure if the methods of making and erecting the precast members are not satisfactory—it is assumed that the high-sounding word " technique " means only mode of production. It is useless to have skilled design and specification without skilful and careful production and erection.

Lectures on Building.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

The Building (Safety, Health, and Welfare) Regulations 1948. New Technical College, Manor Croft, Burton-on-Trent. January 15. 7.15 p.m.

Safety in the Building Industry. By J. A. Hayward. Technical College, St. George Gate, Doncaster. January 15. 7.15 p.m. Also College of Technology, Salmon Pastures, Warren Street, Sheffield. January 24. 7.15 p.m.

Essentials of Good Concreting. By E. E. H. Bate. Municipal College, Victoria Circus, Southend. January 15. 7.30 p.m.

Bound Volumes of " Concrete and Constructional Engineering "

BINDING cases for annual volumes of " Concrete and Constructional Engineering " can be supplied in cloth-covered boards lettered in gold on the spine with the title, volume number, and year of publication. Copies for binding should be sent post paid to Concrete Publications Ltd., 14 Dartmouth Street, London, S.W.1. Where possible, missing numbers will be supplied at the published price to make up incomplete sets, but as many of the numbers published during the past few years are not available it is advisable to ask the publishers whether they have the numbers required before sending incomplete sets. The cost of cloth-covered lettered cases is 6*s.* 3*d.* for each volume. The cost of supplying a case and binding a volume is 15*s.* 3*d.* including packing and carriage.

Recent Developments in Prestressed Concrete. By S. C. C. Bate. Technical College, Great Horton Lane, Bradford. January 16. 7.15 p.m.

Arbitration and Awards. By John J. Clarke. Technical College, Cauldon Place, Stoke-on-Trent. January 16. 7.15 p.m.

Introduction to Prestressed Concrete. By R. C. Blyth. School of Technology, Maidstone Road, Chatham. January 17. 7.15 p.m.

Application of Soil Mechanics in the Design of Foundations. By H. B. Sutherland. North British Hotel, Edinburgh. January 17. 7.15 p.m.

Weathering and Deterioration of Concrete and Cement Renderings. By C. Hobbs. Technical College, Arragon Road, London, E.6. January 17. 7 p.m.

Maintenance of Builders' Plant. By J. Stafford. Technical College, Avenue Road, Grantham. January 21. 7.15 p.m.

Dampness in Buildings. By J. P. Latham. Mining and Technical College, Library Street, Wigan. January 21. 7.30 p.m.

Control of Concrete Quality on Sites. By R. A. Kenny. College of Art and Technology, The Newarke, Leicester. January 22. 7.15 p.m.

Problems of Plastering and Rendering. By E. L. Westbrook. Council Offices, Aberystwyth. January 22. 7 p.m. Also Trafalgar Institute, Charles Street, Milford Haven. January 23. 7 p.m.

Economics of Multi-story Flats Design. By C. N. Craig. Ministry of Works, Ashley Street, Bristol Street, Birmingham. January 23. 7.15 p.m.

Foundation Problems. By N. H. Buchi. Cleveland Scientific and Technical Institute, Middlesbrough. January 29. 7.0 p.m.

Pipes and Pipe Laying. By. N. W. B. Clarke. Technical College, Wulfruna Street, Wolverhampton. January 30. 7.15 p.m.

Arbitration in Building Disputes. By Norman P. Greig. College of Art, Green Lane, Derby. January 31. 7.15 p.m.

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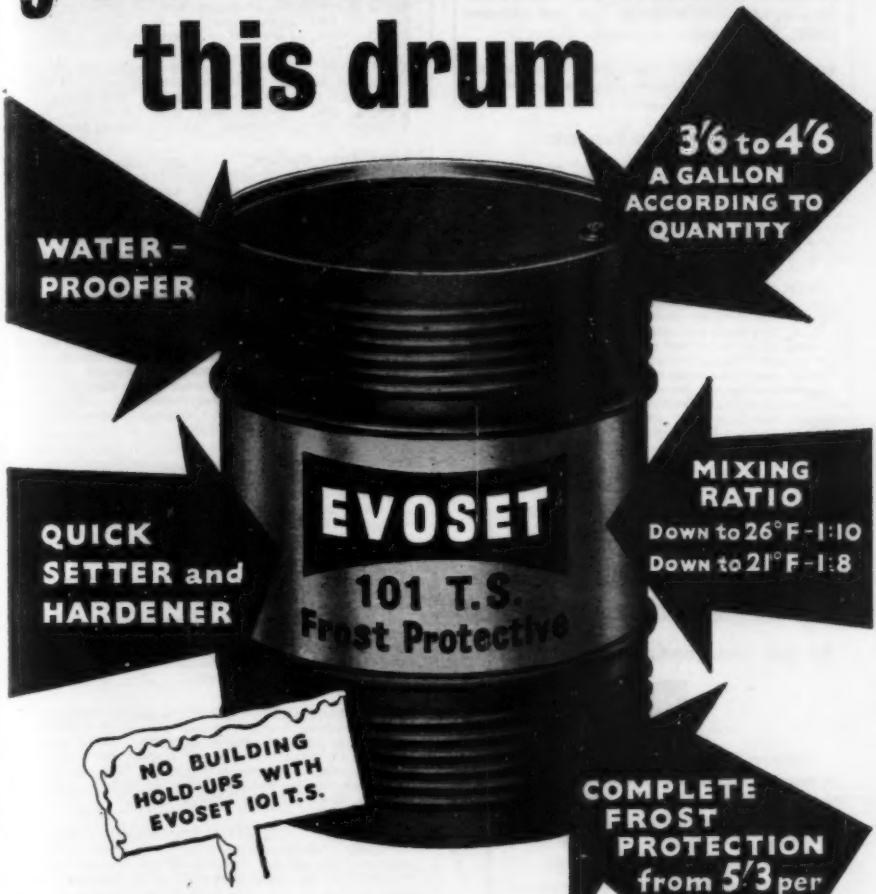
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SITUATIONS VACANT. Experienced reinforced concrete detailers required for interesting and varied work with a large firm of building and civil engineering contractors. 5½-days' week. Canteen facilities. Apply, giving full details of experience, age, and salary required, to Box 4324, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

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SITUATION VACANT. Young engineer or designer, with experience of reinforced concrete design, required for London office of civil engineering contractor. Work will mainly involve calculation and preparation of project schemes under supervision of qualified engineer. Design experience or interest in prestressed concrete an advantage, but not essential. Telephone Victoria 4274 (ext. 12) for appointment, or write to THE LONDON FERRO-CONCRETE CO. LTD., 247 Vauxhall Bridge Road, London, S.W.1. Good opportunity for a suitable applicant. Bonus and pension schemes apply to this appointment.

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SITUATION VACANT. Technical officer. A position is available in the Midlands area for a trainee-engineer interested in concrete technology and prepared to study engineering or science course. Permanent appointment on graduation, with excellent opportunity for advancement. Written application to READY MIXED CONCRETE LTD., 19 Dartmouth Street, Westminster, S.W.1.

SITUATIONS VACANT. TAYLOR WOODROW CONSTRUCTION, LTD., offer first-class prospects for active and experienced site civil engineers, structural design engineers, and soil mechanics geologist engineers. Apply to S.E., Ruislip Road, Southall, Middlesex.

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Civil Engineering Draughtsmen and Designer Draughtsmen are required at the Divisional Drawing Office of the National Coal Board at Tredomen Office, Ystrad Mynach, Hengoed, Glam. Applicants must have experience in one or more of the following branches:

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Starting salaries will be fixed according to age and experience and there will be a site allowance for the third appointment. Pension scheme, luncheon vouchers and 5-days' week are applicable to the London Office appointments.

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(Continued on following page.)

MISCELLANEOUS ADVERTISEMENTS.

(Continued from previous page.)

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CIVIL ENGINEER with at least 2 years' design office experience on structural work and general building construction. Applicants should have passed some part of the qualifying examination of a recognised professional institution or be preparing for such examinations. Scale £500–£1000 with a minimum of £650 at age 25. (*X.531/G.*)

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Write, stating age, education, qualifications and experience, to National Coal Board, Staff Dept., Hobart House, London, S.W.1, *marking envelope with relevant reference number*, before 22 January, 1957.

SITUATION VACANT. Assistant resident engineer required for major road and bridge contract in London area. Experience of reinforced and prestressed concrete bridge construction, earthworks, and concrete roads, advantageous. Salary up to £1,200 per annum. Write, stating age, qualifications and details of experience, to Harry Brompton, Consulting Engineer, 7 Hobart Place, Westminster, S.W.1.

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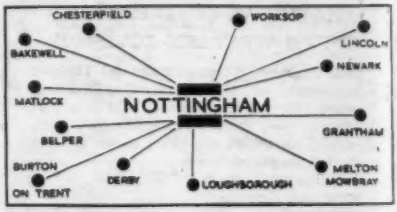

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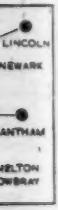

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